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ANALYSIS OF THE UNDERGROUND POWERHOUSE ARCH AT THE PORTAGE MOUNTAIN DAM

A THESIS

SUBMITTED TO THE FACULTY OF GRADUATE STUDIES

IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR THE DEGREE

OF MASTER OF SCIENCE

DEPARTMENT OF GEOLOGY

by

ALAN SHIELDS IMRIE, B.Sc., P. Eng.

EDMONTON, ALBERTA

May, 1967

UNIVERSITY OF ALBERTA

FACULTY OF GRADUATE STUDIES

The undersigned certify that they have read, and recommend to the Faculty of Graduate Studies for acceptance, a thesis entitled "Analysis of the Underground Powerhouse Arch at the Portage Mountain Dam", submitted by Alan Shields Imrie, B.Sc., P. Eng., in partial fulfilment of the requirements for the degree of Master of Science.

ABSTRACT

The powerhouse at the Portage Mountain dam is one of the largest single underground excavations in North America, being 890 feet long, 150 feet high and 67 feet wide. It has been excavated with its long axis parallel to the strike in gently dipping sandstones, siltstones, shales and coals of the Dunlevy Formation.

The powerhouse roof was designed to be a semi-elliptical arch in cross-section and to be supported by a pattern of grouted rock bolts spaced five feet apart. Rock mechanics studies used in the design of the arch and conducted prior to its excavation suggested the presence of residual stresses in the surrounding bedrock. During excavation of the powerhouse arch, significant downward movements of the overlying rock were recorded by a number of different methods. On the basis of these measurements supplementary rock bolting was installed including a change in the basic pattern to a four foot spacing.

Most of the movement is postulated to have occurred by opening of horizontal tension fractures due to blasting. The change in the basic spacing of the rock bolts was unnecessary. It is suggested that a rock bolt pattern which includes the immediate installation of wire mesh at the time of excavation would eliminate extensive supplementary rock bolting. It is further suggested that the excavation of a semi-elliptical arch in thinly-bedded sedimentary rocks is inadvisable.

ACKNOWLEDGEMENTS

The guidance and patience of Dr. L.T. Jory, senior engineering geologist for I.P.E.C. who supervised the writer's work at the Portage Mountain dam, are gratefully acknowledged. To Dr. V. Dolmage, who clearly pointed out the significant role of geology in civil engineering projects, and to Dr. D.D. Campbell, who suggested the thesis problem, special thanks are given. Acknowledgements are also due to the staff of I.P.E.C., especially E.A. Shabatura, whose comments on civil engineering have been very helpful.

At the University of Alberta the writer would like to express his appreciation to his supervisor, Dr. H.A.K. Charlesworth, of the Department of Geology, and to Professor T.H. Patching of the Department of Mining and Metallurgy, whose critical comments and suggestions guided the writer. During the writing of this thesis, the author was a Graduate Teaching Assistant in the Department of Geology.

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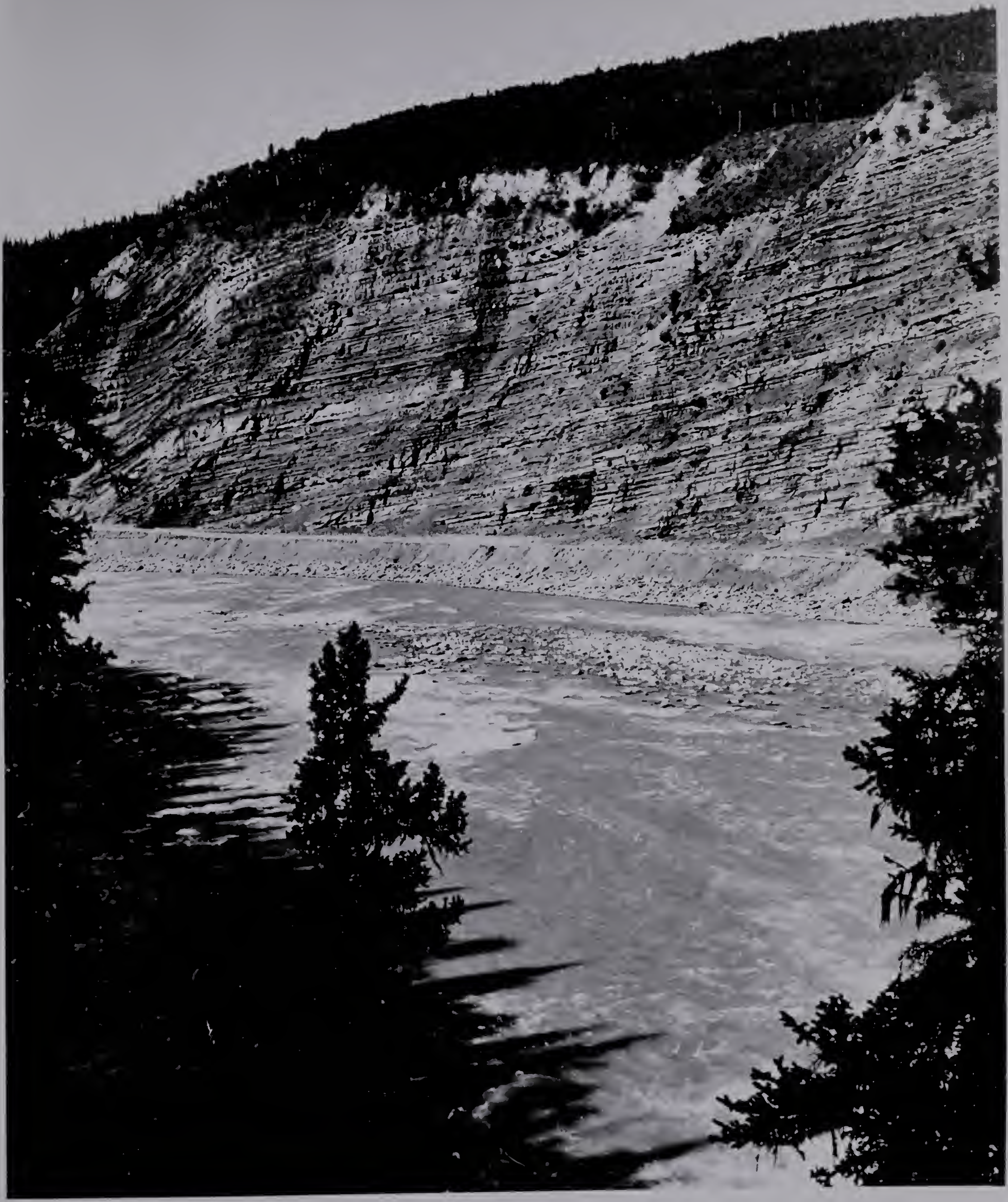


INTRODUCTION

The Peace River Project

Although hydro-electric development of the Peace River in British Columbia (Fig. 1) was envisaged in 1912 (Galloway, 1963, p. 300), the first steps towards harnessing its power were not taken until 1957. Between 1957 and 1959 the Peace River Power Development Company studied the feasibility of constructing one or more hydro-electric dams on the Peace River between Finlay Forks and Hudson Hope (Fig. 2). International Power and Engineering Consultants (I.P.E.C.) were responsible for the engineering aspects of this study while Dolmage and Campbell, consultants, were responsible for the geological investigations.

Geological mapping, diamond drilling and seismic studies were used to investigate 11 sites (Fig. 2). In 1959 the final report of the Peace River Power Development Company recommended building two dams in the Peace River Canyon - a 600 foot high "Portage Mountain Dam" and a 140 foot high "Site I Dam". These sites were chosen essentially on the basis of geology.

The six sites within or near the Rocky Mountains (sites Ursula, 4,5,6,7,8) were all found to be unsatisfactory. Diamond drilling revealed a deep narrow canyon below the present floor of the river in this region filled with permeable alluvium. Of the five sites in the Peace River Canyon, the Portage Mountain Site and Site I were regarded as the most satisfactory from the standpoints of foundation rock suitability and economics of construction. The report recommended that the glacial outwash deposits in Portage Pass (Fig. 2) be used in constructing the gravel-fill Portage Mountain Dam.

In 1961 the Peace River Development Company was taken over by the government-controlled British Columbia Hydro and Power Authority. I.P.E.C., also government-controlled with the takeover, retained responsibility for the engineering

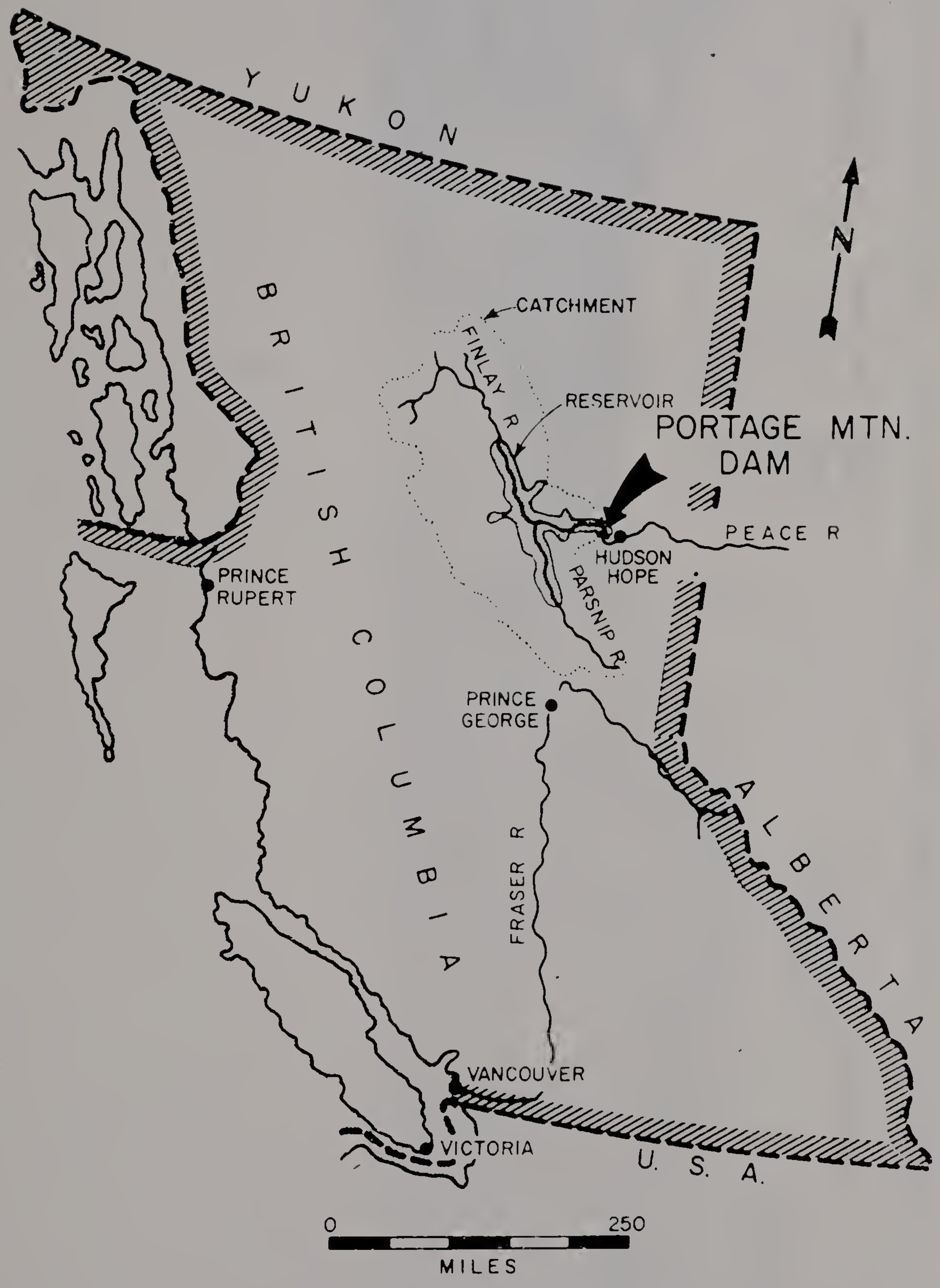


FIG. 1 - LOCALITY PLAN

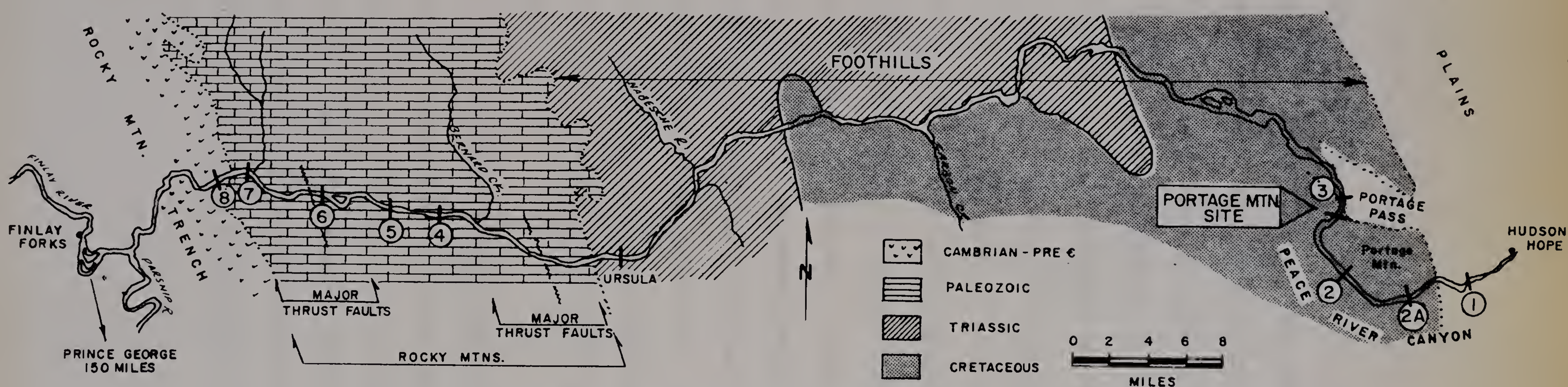


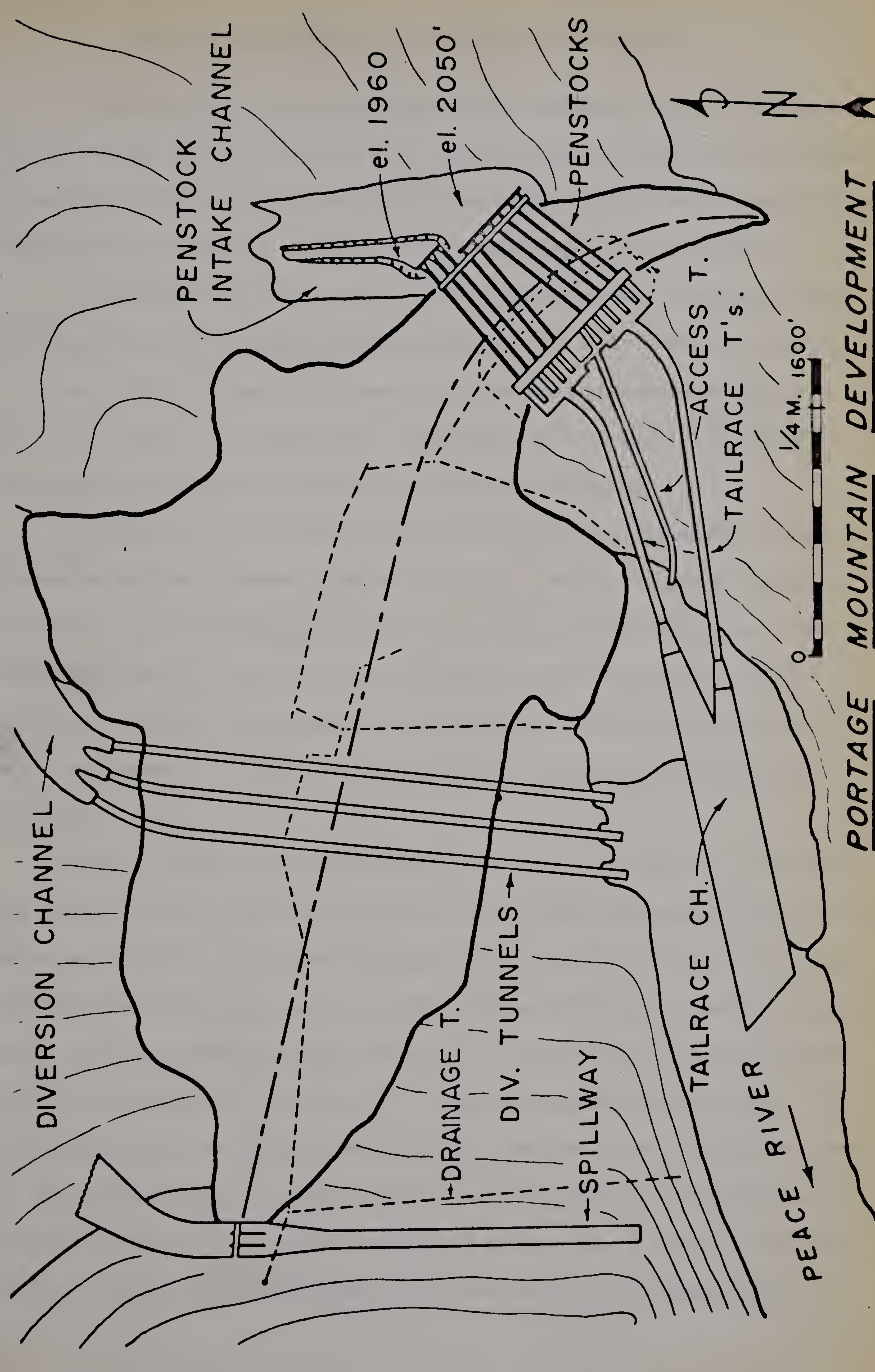
FIG. 2 GEOLOGY AND DAMSITES ALONG THE PEACE RIVER, FINLAY FORKS TO HUDSON HOPE
(after Dolmage & Campbell, 1963)

design of the Peace Power Project.

In April 1962 construction at the Portage Mountain damsite began with the award of an $18,000,000 contract for the building of three, 48-foot-diameter diversion tunnels within the right abutment (Fig. 3). These were completed and the river was diverted in September 1963. At this time a $73,000,000 contract was awarded to a consortium of contractors for foundation preparation and subsequent construction of the dam. The dam will be completed in September 1967.

Because a surface powerplant would have required 2500-foot-long penstocks, an underground powerplant under the left abutment of the dam with 800-foot-long penstocks appeared to be more economical to build. Experience gained from excavating the diversion tunnels suggested that the rock surrounding the powerhouse would require relatively minor support. After a decision to proceed with the underground powerplant was made in early 1963, Professor N. Hast of Sweden was asked to measure the stresses in the vicinity of the proposed powerplant before excavation began. This knowledge of the stress situation was to be used to ensure a safe and economic design of the powerhouse and manifold structures. However, the final results of Professor Hast's work, along with subsequent photo-elastic strain measurements by C-I-M Consultants, were not available until after the basic design of these structures had been completed.

Construction began on the $77,000,000 powerplant contract in July 1965, and the first power is expected to be generated in October 1968. In the summer of 1966, a $43,000,000 contract was let for the construction of the spillway on the upper right abutment of the dam. Also in the summer of 1966 the second phase of the Peace Power Project was initiated with detailed exploration of Site 1 fourteen miles downstream. The total cost of the Peace Power Project will exceed $750,000,000.



PORTAGE MOUNTAIN DEVELOPMENT

PLAN

FIG. 3

Technical Description of Portage Mountain Development

The Peace River above the main dam has a catchment area of 27,800 square miles. Run-off water from this area will be ponded behind the dam forming a reservoir extending 65 miles west to the Rocky Mountain Trench at Finlay Forks, then 150 miles along the Trench (Fig. 1). This will be the largest lake in British Columbia.

The dam itself will be a zoned, gravel-fill embankment, 600 feet high and 6700 feet long. The maximum width of the dam will be about 3000 feet narrowing to a crest width of 30 feet. The slopes of the upstream and downstream faces will be 2.5 to 1 and 2 to 1, respectively. Large blocks of sandstone will be placed on the upstream and downstream faces as rip-rap to prevent erosion.

Before any fill was placed in the old river-bed and on the abutments, the foundation was well prepared. The entire dam area was first stripped to bedrock. Below the core of the dam (zone 1) dental concreting, rock bolting and guniting were then carried out where necessary, followed by blanket grouting to a depth of 30 feet. Finally a much deeper curtain grouting (200-300 feet) was completed within the bedrock on a 60-foot-wide strip paralleling the axis of the dam below the core.

Figure 4 shows a cross-section of the dam in the river-bed portion illustrating the different zones of gravel-fill as well as the two patterns of grouting. Zone 1 will be an impervious silty sand containing about 28 per cent silt. The relatively narrow transition and filter zones, zones 2 (gravelly sand) and 3 (fine sandy gravel) respectively, will be situated downstream of the core to prevent zone 1 from washing into the free draining zone 4 (coarse gravel). Zone 6, composed of sandy gravel, will be a random shell zone located both downstream from zone 4 and upstream from zone 1. The function of zone 6, which will make up about half of the entire volume of the dam, will be to keep the entire assemblage stable. Zone 5, a very coarse gravel, will be located on the upstream side of the dam to allow free drainage during

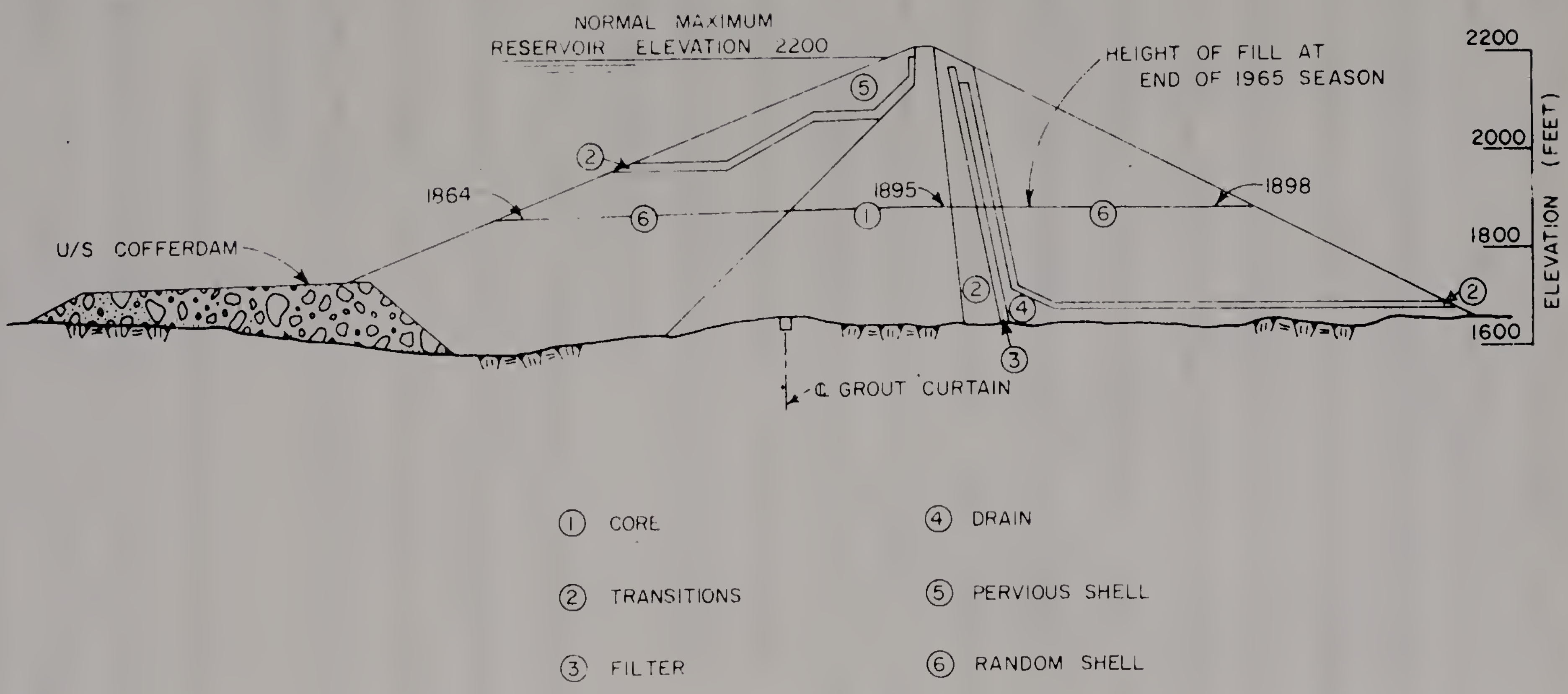


FIG. 4 - DAM SECTION

reservoir draw-down.

The source area of the gravel-fill for the embankment is the southern of two separate outwash deposits located about four miles east of the site in Portage Pass. A conveyor belt 15,000 feet long and driven by four 850 horsepower electric motors delivers the outwash material to a processing plant near the dam on the left abutment. In the processing plant the silty and sandy gravels are screened and washed to produce different sized material as required for the various zones described above. Lacustrine silt in a deposit adjoining the processing plant has to be blended with sand from the outwash deposit to produce material for zone 1. The processed material is then conveyed to a special loading system at the edge of the dam where it is transferred into 100-ton bottom-dumping trucks. These huge machines transport the material to the various zones of the dam where it is dumped, spread and compacted. During the rather limited fill placement season when the weather is above freezing (mid-April to October) the dam rises almost 1 1/2 feet per day. In terms of volume this placement rate amounts to 3,000,000 cubic yards per month. The total volume of the dam will be 58,000,000 cubic yards.

In the event that some water does manage to seep through the core or the grout curtain, a series of drainage tunnels has been driven in the bedrock under both abutments of the dam (Fig. 3). These tunnels, assisted by connecting drain holes, will intercept and channel any seepage water thus preventing any dangerous increase in uplift pressures.

The powerplant, which is at present under construction, is located within the left abutment of the dam (Fig. 3). Figure 5 provides a typical cross-section of the various structures involved in the generation of power. Water will enter the intakes at the upstream toe of the left wing of the dam before flowing down ten 18-foot-diameter penstocks to the turbines in the lower part of the powerhouse chamber. The chamber itself is 890 feet long, 150 feet high and 67 feet wide. Ten 310,000

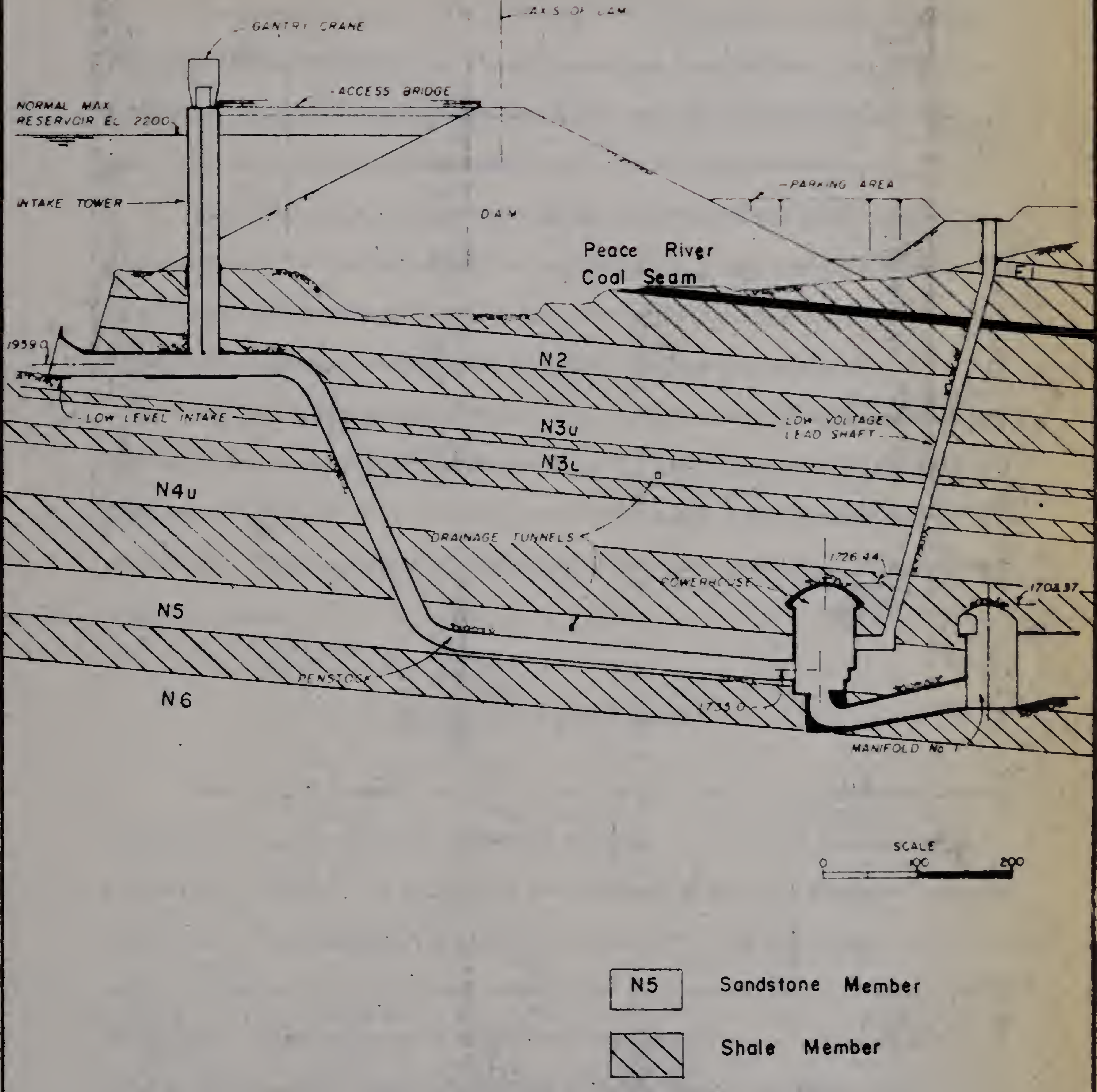


FIG. 5 TYPICAL GEOLOGICAL SECTION POWERPLANT

horsepower Francis turbines running at a speed of 150 rpm will drive 227 megawatt generators.

The generated power, at a relatively low 14,000 volts, will be carried up the lead shafts to the surface switchyard downstream from the dam where it will be transformed to 500,000 volts. From here the power will be transmitted south to a power grid extending throughout central and southern British Columbia.

Water after going through the turbines will pass into the draft tubes before discharging into two large manifolds or surge chambers. Each manifold is connected to a 71-foot-high horseshoe-shaped tailrace tunnel which will convey the water to an excavated tailrace channel in the river-bed downstream from the dam.

The final major structure at the damsite to be constructed is the spillway channel at the top of the right abutment (Fig. 3). This structure, 100 feet wide and 2300 feet long, will spill any excess reservoir water above that handled through the powerplant. The entrance to the spillway will be a gate-controlled, 800-foot-long approach channel.

Statement of Thesis Problem

This thesis is a partial analysis of the design of the Portage Mountain Development powerhouse arch using information available to the design engineers prior to excavation of the arch and information recorded both during and after completion of excavation. This information includes the results of the rock mechanics studies carried out to investigate the *in situ* stress conditions of the bedrock and the results of strain measurements in the rocks above the arch during construction. A detailed examination of these results is used to help assess the basic support and shape of the powerhouse arch as originally designed.

The data for this thesis was obtained while the writer was employed by I.P.E.C. as assistant engineering geologist during the period May 1964 to September 1966.

GENERAL GEOLOGY

Geomorphology

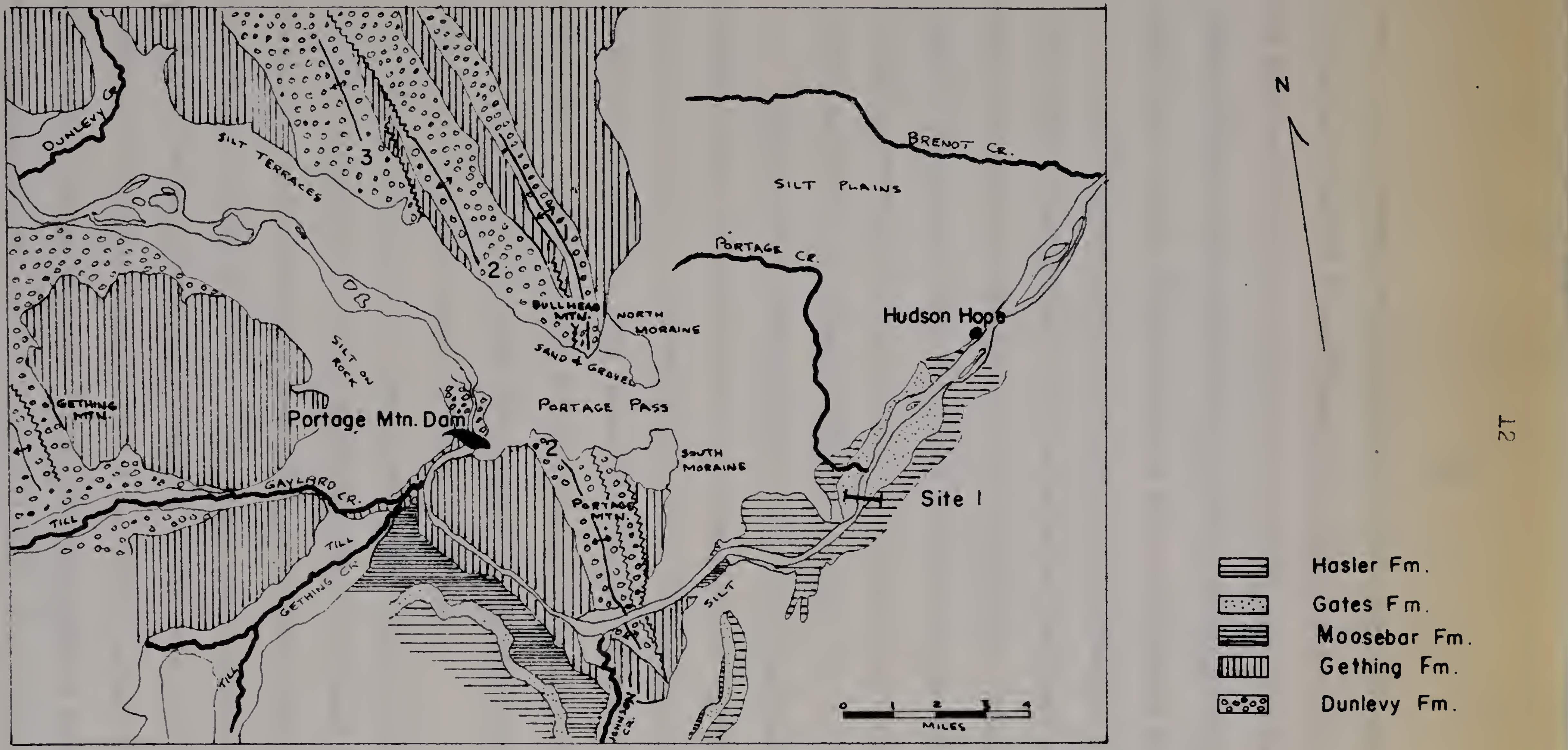
General Physical Features

The Portage Mountain damsite, one mile below the head of the Peace River Canyon, is situated close to the eastern border of the Rocky Mountain Foothills which are 40 miles wide at this latitude (Fig. 2). The Interior Plains to the east consist of fairly high, rolling country mainly underlain by Upper Cretaceous sediments. To the west the Main Ranges of the Rocky Mountains are separated from the Western Cordillera by the Rocky Mountain Trench. At Finlay Forks where the southward flowing Finlay River and the northward flowing Parsnip River join to form the Peace River, the Trench is almost 8 miles wide.

The Peace River is generally quiet flowing and navigable between Finlay Forks and the Portage Mountain damsite. The average gradient in this region is three feet per mile with an average river width of 800 to 1000 feet. Approximately one mile above the damsite, however, the Peace River abruptly swings south and then east around Portage Mountain in a deep, narrow canyon 17 miles long with walls 200 to 700 feet high (Fig. 6). The width of the Peace River at the head of the Canyon is only 250 feet and the water elevation is 1690 feet above sea level. Seventeen miles downstream where the river leaves the Canyon near Hudson Hope the Peace River has widened to 900 feet and has fallen some 200 feet in elevation. The average gradient of the river in the Canyon is about 12 feet per mile. Downstream from Hudson Hope the river remains at least 800 feet wide in a post-glacial valley two to three miles wide.

Formation of Peace River Canyon

The pre-glacial Peace River flowed between Bullhead and Portage Mountains through what is now known as Portage Pass. A number of vertical test holes have



after Dolmage & Campbell (1963)

FIG. 6 GEOLOGY of the PEACE RIVER CANYON

been drilled in Portage Pass in connection with construction material investigations for the dam. None of these hit bedrock but instead penetrated mainly clays, silts and fine sands of lacustrine origin. One test hole went 1738 feet deep or approximately 480 feet above present sea level without hitting bedrock. Three separate gravels approximately 50 feet thick were intersected by some of these test holes.

The Pleistocene Epoch saw this portion of northeastern British Columbia caught between two ice fronts: the Laurentide and the Cordilleran ice sheets advancing from the northeast and the west, respectively. Although the regional extent of these two ice fronts is not fully known, the Laurentide glaciation probably did not extend to Portage Mountain while Cordilleran ice may have reached 40 miles east of Portage Mountain (Mathews, 1962).

Ponding of melt water between these two ice masses resulted in the formation of glacial lakes at various intervals with water elevations determined in part by the relative locations of the two major ice sheets. Two important lake elevations are indicated by strand lines at the present 2750 foot and 2260 foot contours east of Hudson Hope. Clays, silts and fine sands deposited in these glacial lakes are widely distributed in the Hudson Hope region; in some cases (Portage Pass) these lacustrine sediments are almost two thousand feet thick.

Towards the end of the Pleistocene the Cordilleran ice reached at least as far east as Portage Pass. Its front apparently remained between Bullhead and Portage Mountains for some time as evidence by two high outwash deposits which extend out from both of these mountains. These deposits are referred to as the North and South Moraines (Dolmage and Campbell, 1963). Melt water from the westerly receding main ice mass became ponded to the east by the lacustrine sediments and ice occupying Portage Pass, and a glacial lake was formed to the west. Terraced drift suggests that this lake had its upper elevation about 2400 feet above sea level and that it extended at least 40 miles west into the Foothills (Mathews, 1947).

At an elevation of 2400 feet the trapped lake waters were able to escape over a bedrock saddle west of Portage Mountain. Canyon excavation, once initiated about 11,000[1] years ago, continued rapidly as the water downcut through Lower Cretaceous sandstones, shales and coal. The Peace River may have usurped the pre-glacial valley of Johnson Creek in the vicnity of the halfway point of the Canyon (Beach and Spivak, 1943; Dolmage and Campbell, 1963).

The preceding discussion on the formation of the Peace River Canyon noted that the lacustrine silts in Portage Pass helped dam a large glacial lake extending west towards the Rocky Mountains. The infilled ancestral valley presents a possible seepage path for the reservoir, and piezometers and stream gauges have already been installed in Portage Pass. Although ground-water pore pressures can be expected to rise as the reservoir fills, the previous success of the material in Portage Pass as a natural dam and the extreme length of the possible seepage path both suggest that there is very little chance of a reservoir failure through Portage Pass.

The North and South Moraines

The North and South Moraines are outwash deposits resting on lacustrine sediments at an elevation of 2400 feet. These deposits are 200 to 300 feet thick consisting of interbedded sand and gravel together with about 12 per cent silt. The beds strike approximately north-south and dip 16 to 20 degrees easterly. Most of the individual particles are well-rounded and three inches or less in size. Their composition confirms a western source. Two mammoth tusks have been unearthed 150 feet down in the South Moraine.

[1] Carbon 14 dating of a mammoth tusk uncovered in the South Moraine in the spring of 1966 gave an age of 11,500 years B. P.

Stratigraphy

The stratigraphy of the Rocky Mountain Foothills in the Peace River Valley is given in Tables 1 and 2. Emphasis is placed on the Lower Cretaceous Bullhead and Fort St. John Groups which crop out in the immediate vicinity of the Peace River Canyon.

Structure

The regional structural trend in the Foothills of Northeastern British Columbia averages N 20° W to N 30° W. Within the Foothills folding is more important than faulting. According to Irish (1962, p. 110)

> "Many southwest dipping thrust faults occur but stratigraphic displacement on any single fault is normally measured only in tens or hundreds of feet. The folds consist of a succession of narrow, compressed anticlines separated by broad synclines having gently dipping limbs.... In many cases the tightly folded anticlines are faulted near their axes. Several anticlines along Peace River Valley have their southwest flanks steeply inclined while the northeast flank is inclined at a very small angle which tends to give the fold a flat-topped appearance."

Beach and Spivak (1944) have remarked that the folds are concentric rather than similar and suggest that " such folds are more superficial than folds of similar size in the southern Foothills." The anticlines and synclines plunge southeast.

The Portage Mountain - Butler Ridge anticlinorium (Beach and Spivak, *ibid*) marks the eastern boundary of the Foothills and is the most important structural feature in the immediate vicinity of the damsite. The anticlinorium strikes 5 to 20 degrees west of north and has a maximum width of three miles.

The easternmost component of this major structure, the Bullhead anticline, is well exposed on the southern face of Bull head Mountain (Fig. 6). It consists of folded Dunlevy sandstone with the east and west limbs both dipping at 60 degrees. On Portage Mountain the east limb of this anticline consists of Gething strata. However, near the axial surface Dunlevy sandstones are thrust over Gething

TABLE 1

Lower and Middle Mesozoic Rocks, Peace River Valley

Series	Group	Formation	Thickness (ft.)	Lithology	Reference
Jurassic	Fernie		1100	Dark marine shales with some sandstone, siltstone and limestone	McLearn and Kindle (1950, p. 60)
Upper Triassic		Pardonet	250-2000	Dark grey silty limestone with minor clastics	McLearn and Kindle (1950, p. 35)
Upper & Middle Triassic		"Grey Beds"	2500	Grey limestones; grey massive calcareous sandstone	
Middle Triassic		"Dark Siltstones"	75-430	Thin bedded calcareous siltstones, dark limestone	
		"Flagstones"	235-380	Grey flaggy, calcareous siltone and sandstone	

TABLE 2

Upper Mesozoic Rocks, Peace River Valley

Series	Group	Formation	Thickness (ft.)	Lithology and Physical Characteristics	Reference
Lower Cret-aceous	Fort St. John	Hasler	700	Marine shales	Beach and Spivak (1944)
		Gates	300	Marine sandstone with interbedded shale	McLearn and Kindle (1950, p. 76)
		Moosebar	1100	Dark marine shale containing minor sandstone and ironstone	McLearn and Kindle (1950, p. 74)
	Bull-head	Gething	1400	Interbedded sequence of non-marine shales, siltstones, sand-stone and bituminous coal. Dark grey to black shales pre-dominate. Ripple marks, cross bedding, facies changes and plant remains common.	McLearn and Kindle (1950, p. 65)
		Dunlevy	3000 - 3200	Sandstone, fine-grained to conglomeratic, quartzose; conglomerate; interbedded shale, locally carbonaceous; non-mar-ine at top grading down to marine sediments at base	Beach and Spivak (1944)

shales. West of the Bullhead anticline is the Bullhead syncline, which is locally faulted on Butler Ridge. Further west the main structural feature of this anticlinorium is the Butler anticline exposed on both Portage Mountain and Butler Ridge. The eastern limb of this anticline on Portage Mountain is broken by a west dipping thrust fault, probably the same fault that cuts the Bullhead syncline on Butler Ridge. The westernmost major structural feature of the anticlinorium is the Danish Creek anticline exposed on Butler Ridge but not readily apparent on Portage Mountain.

Most of the individual structures comprising the Portage Mountain - Butler Ridge anticlinorium can be followed northwest of Bullhead Mountain for about nine miles. South of Portage Mountain and Mount Johnson the anticlinorium plunges southwards.

West of the Portage Mountain - Butler Ridge anticlinorium the broad shallow Dunlevy syncline is the next important structural feature. Gething strata are exposed in the axial zone of the syncline. West of the Dunlevy syncline, trending N 20° W through Mount Gething, is the Gething - Stott structural zone (McLearn and Kindle, 1950, p. 114). This anticlinal feature consists of a wide (one mile) flat crest with a steep southwest limb and a steep northeast limb cut by a thrust fault. Northwest of Mount Gething where older beds are exposed at lower elevations the anticline becomes narrower. This suggests concentric folding which has also been noted by Mathews (1947) in the Carbon Creek Basin.

Similar structural features are found in the western Foothills but are not discussed here. East of the Foothills in the Interior Plains, generally flat-lying Cretaceous strata are exposed in the Peace River Valley. Some minor thrust faults with displacements of up to a few tens of feet and a few very gentle folds do exist.

Damsite Geology

This section describes in detail the physical features, stratigraphy and structural geology of the damsite as derived from diamond drilling, surface mapping and underground mapping during the period 1958 to 1966. Emphasis is placed on the geology of the powerhouse arch.

Physical Features

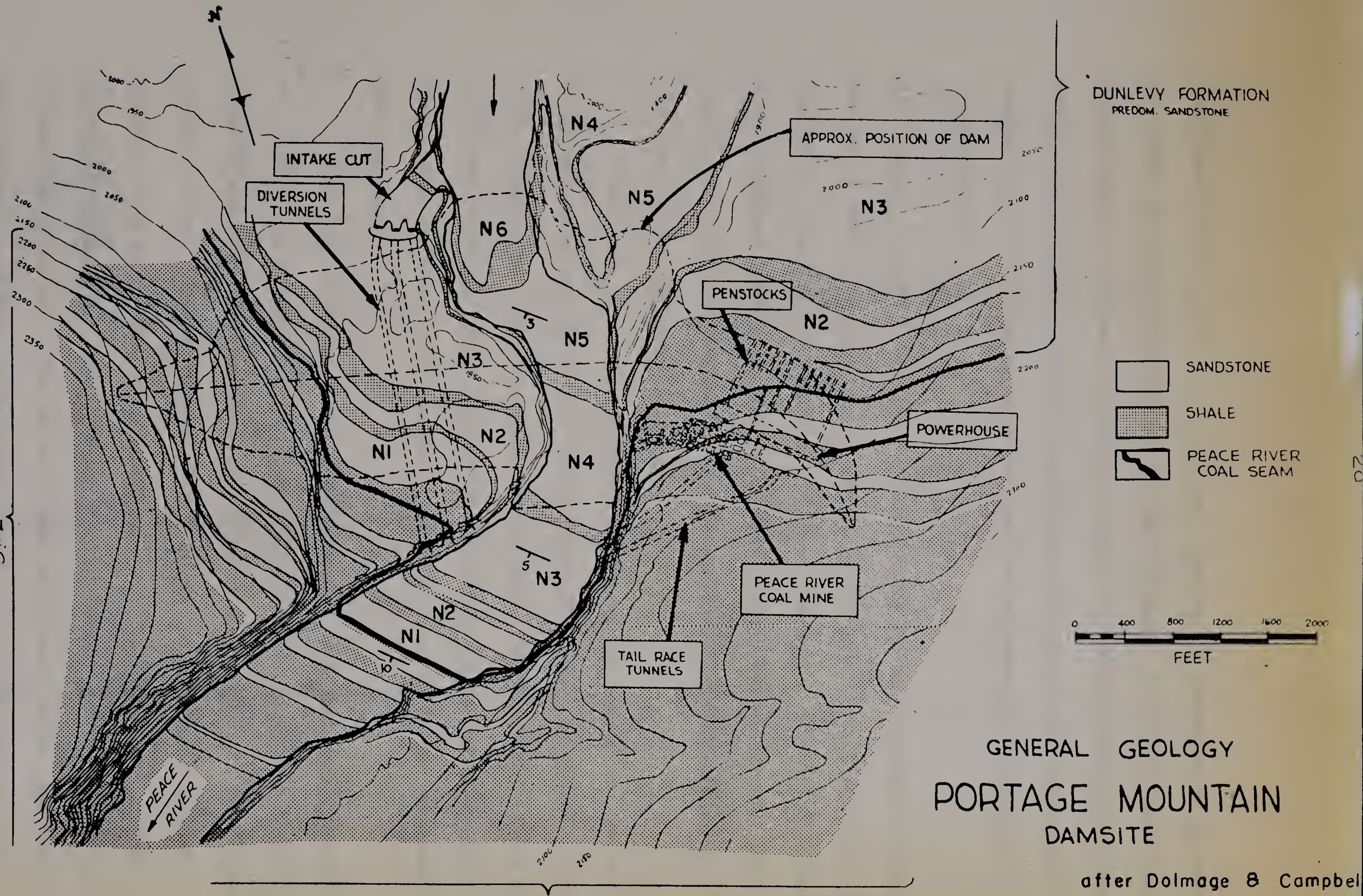
The dam is situated one mile below the entrance to the Peace River Canyon at a point where the river makes a modified S-bend (Fig. 7). The river averages 600 to 700 feet in width between a right canyon wall 150 feet high and a left canyon wall 300 feet high.

From the top of the left canyon wall the ground rises to the top of Portage Mountain. On the right bank a triangular bedrock plateau, 3500 feet by 2000 feet and 150 feet above the river, underlies part of the dam. Beyond this the ground rises past the crest elevation of 2230 feet.

Stratigraphy

Strata belonging to the Dunlevy and Gething Formations underlie the damsite (Fig. 7). The base of the Peace River Coal Seam (Dolmage and Campbell, 1963) is taken to be the contact between these formations. This excellent marker bed, averaging seven feet thick, was mined by the Peace River Coal Mining Company until 1949. The workings of the old mine, which have been backfilled, underlie the dam at the 2000 foot level on the upper left abutment.

Lithologically, Dunlevy and Gething strata both consist of interbedded Lower Cretaceous sandstones, shales and coals of continental origin. Sandstone predominates in the Dunlevy Formation whereas shale is most common in the Gething Formation. The major sandstone members within the Dunlevy Formation are designated by the prefix "N" and are numbered down from the Peace River



GETHING FORMATION

Figure 7

Coal Seam (N1, N2, N3, ...). The major Gething sandstones are designated by the prefix "E" and are numbered up from the coal seam. Further division of sandstone members into 'lower', 'middle' and 'upper' units is also made when necessary. Major shale members are given the same designation as the underlying sandstone member (e.g. the N4 Shale lies between the N4 Sandstone below and the N3 Sandstone above).

(i) Dunlevy Formation. The upper 450 feet of the Dunlevy Formation has been mapped in detail at the damsite and consists of massive sandstone beds, 20 to 100 feet thick, separated by much thinner shale and coal sequences (Table 3). The strata contain or exhibit such features as plant remains, fossil tree trunks, dinosaur tracks, lenticular shale and coal horizons and numerous local unconformities, all characteristic of continental deposition.

The fine to very coarse-grained, "salt-and-pepper" sandstone contains 50 to 70 per cent quartz, 15 per cent "chert", 20 to 30 per cent shale particles and minor carbonate and clay minerals (Dolmage, 1965). Each sandstone member rests with sharp contact on coal or coaly shale. Conglomeratic beds grade upwards into medium and fine-grained sandstone, and this in turn grades into the overlying shale member.

The shale members generally grade from very sandy shale and siltstone at the base to coaly shale and coal at the top. These members range from 5 to 30 feet thick although the N5 Shale thickens to 70 feet in the powerplant area because of a facies change. The shales consist of 50 to 75 per cent quartz, 10 per cent feldspar, 10 to 20 per cent chlorite and kaolinite, 5 per cent chert and minor carbon (Dolmage, *ibid*). The shales are generally massive, very fine-grained, brownish-black and soft. They do not compact significantly when subjected to high loading, nor is there any noticeable swelling observed if the shales are saturated with water. However, they tend to weather strongly when exposed

TABLE 3

Stratigraphy and Lithology of the Upper 450 Feet of the Dunlevy Formation at the Portage Mountain damsite.

on	Member	Thickness	Lithology and Physical Characteristics
ng	Peace River Coal Seam	7 ft.	High-grade, low-ash, bituminous coal with a 6-inch clay-ironstone bed in the middle.
	N1 Shale	10 ft.	Carbonaceous shale with minor amounts of coal.
	N1 Sandstone	25 ft.	Fine-grained, medium-grey sandstone; about 25 feet thick where exposed on right abutment plateau, it begins to shale out at the canyon edge of the plateau and is not present on the left abutment.
	N2 Shale	20 ft.	Dark grey to black sandy shale with minor amounts of coal and coaly shale. About 20 feet thick on the right abutment, it thickens to 55 feet on the left abutment where it includes the equivalents of the N1 Shale and Sandstone.
	N2 Sandstone	25 ft.	Massive, medium-grained, white, rarely jointed sandstone. Its lower contact is sharp and its upper contact gradational to sharp.
	N3 Shale	35 ft.	Interbedded shale, sandy shale and siltstone with 6 inches to 1 foot of coal at the top; the shales are fractured and broken in places; two to three inches of coaly gouge underlie the coal seam.
	N3 Sandstone	80 ft.	Fine to medium-grained, dirty sandstone containing lenticular shaly horizons and an intraformational breccia; generally it is divided into two sandstone units ($N3_U$ and $N3_L$) separated by 0 to 12 feet of shales. The sandstone is a competent rock with few natural fractures.
	N4 Shale	20 ft.	Interbedded sandy shale and very fine-grained sandstone with 1 to 3 feet of coal, locally sheared at the top.
/	N4 Sandstone	85 ft. 50 ft.	Right abutment - fine to coarse-grained, competent sandstone. Divided into Upper and Lower sandstone units separated by 3 to 8 feet of shale. Lower unit shales out to NE. Left abutment - medium to coarse-grained, medium-grey, competent sandstone with occasional shaly partings; it is equivalent to the $N4_U$ Sandstone of the right abutment.
	N5 Shale	35 ft. 70 ft.	Right abutment - interbedded dark brownish-grey siltstone and shale with numerous thin coal seams throughout and a prominent 18-inch coal ("springline coal") at the top; a continuous 2-inch gouge lies 2 feet below the main coal seam. Left abutment - interbedded shale and siltstone with 9 coal or coaly shale seams and 3 gouge horizons; the increase in thickness is caused by its having incorporated the shale equivalent to the $N4_L$ Sandstone
	N5 Sandstone	50 ft.	Medium to coarse-grained sandstone with conglomeratic horizons towards the base; shaly partings are common locally.
	N6 Shale	50 ft.	Predominantly sandy shale commonly diffusely interbedded with shaly sandstone; the upper 5 to 10 feet consists of coal, coaly shale and shale with 1 to 2 inches of gouge from the top.

to alternating periods of wetting and drying or freezing and thawing.

The well-cleated, bituminous coal seams that normally occur at the very top of shale member are up to 24 inches thick. River undercutting, a common feature in the Canyon, is usually associated with the easily erodable coal horizons.

(ii) Gething Formation. The Gething Formation crops out only on the upper portions of the right and left abutments (Fig. 7). Lithologically, the Gething and Dunlevy strata are very similar and a description of the Gething rock types would be repetitious The main difference between the two formations is found in the percentage of each rock type present: shales and coal make up 50 and 5 per cent respectively of the Gething strata and only 30 and 2 per cent of the Dunlevy Formation. The remaining 45 per cent of the Gething section consists of sandstones and siltstones, whereas almost 70 per cent of the exposed Dunlevy Formation consists of massive sandstone.

The Gething sandstone and shale members, where mappable within the damsite, are usually less than 25 feet in thickness. Sharp facies changes make even thick sandstone beds difficult to trace for long distances. The overall strength of the Gething Formation is less than that of the Dunlevy but Gething strata underlie only the upper portions of the dam where reservoir and foundation pressures are low.

Structural Geology

At the damsite Dunlevy and Gething strata, which comprise part of the western limb of the Butler anticline, strike N 47° W and dip gently southwest. Dips are five degrees on the left abutment and increase to ten degrees on the down-stream side of the right abutment. No major fault is present at the site. Bedding-plane slip associated with folding apparently led to the development of gouge seams in various shale and coal horizons. These thin (1/8 to 3") seams parallel the bedding and present possible zones of slip when the reservoir is filled. Other tectonic features include local monoclinal warps, well-developed cleating in the

coal and poor to well-developed jointing in the siltstone and shale. Shearing is common at the base of the thicker coal seams. Joints are generally absent in the massive sandstones.

Two different geological structures, probably formed as a result of erosional unloading during the formation of the Canyon, are present in the vicinity of the river channel. Fractures parallel to bedding and up to a foot wide were uncovered in the bedrock floor of the river during foundation preparation. These openings, filled with alluvium, probably formed due to the buckling of the strata under the influence of horizontal stresses in the bedrock. Vertical sheeting parallel to the canyon walls is found on both abutments but the feature extends only a few tens of feet away from the canyon.

Powerplant Geology

The powerhouse and its ancillary works (manifolds, penstocks, tailraces, low voltage lead shafts) are being constructed underground below the left abutment of the dam. Some 300 to 400 feet of bedrock overburden overlies most of the powerhouse structures. Virtually all of the rock excavation required for these structures is in the top 450 feet of the Dunlevy Formation (Fig. 5, Table 3).

The ten 23-foot-diameter penstock tunnels were driven 400 feet up dip in the N5 Sandstone from the powerhouse before rising through N5, N4, N3 and N2 members to the intakes. The powerhouse and both manifolds are oriented so that their long axes parallel the strike of bedding which dips about five degrees southwest in this vicinity. The upper 65 feet of the powerhouse and manifold chambers are within the N5 Shale. The base of the manifolds are in the N5 Sandstone but the machine chamber passes through the sandstone to include 40 feet of the N6 Shale.

The top of the powerhouse arch is 16 feet below the base of the N4 Upper Sandstone. The manifold arches, similar in size but lower in elevation, are

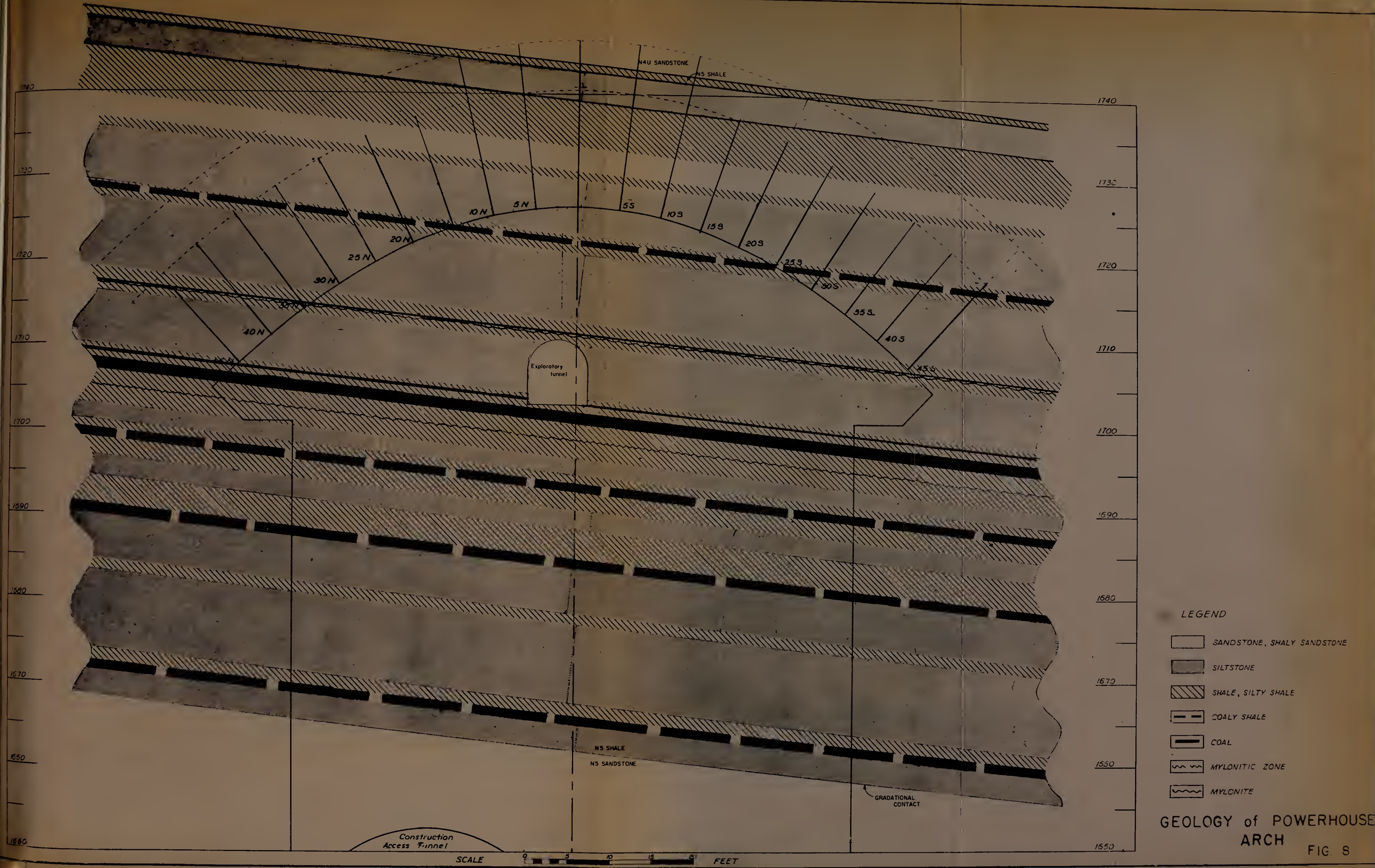
located in essentially the same strata because of the dip of the bedding. Figure 8 is a cross-section of the upper part of the powerhouse looking southeast along strike from the northwest end of the chamber. Samples of the sandstone, siltstone and shale rock types indicated on this section were sent to Dolmage and Campbell for thin section examination.

The N4 and N5 Sandstones, both very similar lithologically, "were found to consist of about 60% quartz, 20% shale fragments and 15% chert. The remainder consists of about equal amounts of calcite and white mica. The grains are tightly packed and intergranular spaces are occupied by chert... (the interstitial) chert was introduced as a cementing material after the lithification of the rock. It is chert cement which gives the rock its great strength, high density and durability and enables it to store the large amounts of energy found by Hast." (Dolmage, 1965).

Siltstone constitutes about 60 per cent of the N5 Shale as seen in Figure 8. Dark brownish-grey in colour it is much finer grained than the sandstone but is still a structurally sound rock. In the absence of sandstone, anchorage for structural rock bolts was attempted in siltstone horizons. Jointing, not well-developed in the shale and sandstone, is best developed in these siltstone beds. The attitude of these joints is approximately N 70° W/vertical and their spacing ranges from one foot to four feet. The siltstone is made up of 85 per cent quartz, 2 to 3 per cent "chert", 10 per cent clay minerals and 1 or 2 per cent carbon. The grains average 0.05 mm. in size with a groundmass probably consisting of clay minerals, carbon, mica and some "chert" (Dolmage, ibid).

The dark grey to black shale is a soft, weak rock not particularly suitable for rock bolt anchorage. The composition is similar to that of the siltstone except that quartz makes up only 50 to 75 per cent of the rock (Dolmage, ibid.).

Well-cleated coal and coaly shale are present throughout the N5 Shale in seams 3 to 18 inches thick. The cleat averages N 35° E/vertical and its spacing ranges from 1/8th inch to 2 inches.



GEOLOGY of POWERHOUSE ARCH

FIG. 8

In addition to these five rock types, gouge seams ranging from a fraction of an inch to over two inches in thickness have been observed at three different horizons in the N5 Shale.

ROCK MECHANICS STUDIES

Introduction

The decision to place the powerplant underground at the Portage Mountain damsite was made early in 1963 and preliminary design was begun at that time. During construction of a structure such as this, one of the main concerns is that stresses in the bedrock surrounding the openings do not exceed the strength of the rock. The presence of high residual stresses in the rock must be taken into account when calculating the increased stress created by the openings.

Since 1932 (Olsen, 1957, p. 186) attempts have been made to measure these residual stresses. Previously the state of stress at some point underground was estimated by using the following relationships which consider only the weight of the overlying rock and which also consider the rock to be perfectly elastic, isotropic and homogeneous:

$$\sigma_v = pgh \qquad (1)$$

and

$$\sigma_h = \frac{\mu}{1-\mu} \cdot \sigma_v \qquad (2)$$

where σ_h, σ_v are the horizontal and vertical stresses at some point P below the surface, pgh is the weight of rock above P, and μ is Poisson's ratio. Assuming Poisson's ratio to be 0.25, the horizontal stress would be approximately 1/3 of the vertical stress.

Within the last ten years refined methods of measuring <u>in situ</u> rock stresses have shown that the above stress relationships are invalid around many underground openings. Results of stress measurements at the Poatina Power Station in Tasmania (Endersbee and Hofto, 1963), Tumut 1 (Moye, 1959) and Tumut 2 (Pinkerton and Gibson, 1964) Power Stations in Australia, and in various mines in Scandinavia (Hast, 1958) have been published. In these cases the horizontal stresses were from four to over ten times that anticipated by applying equation (2) above.

With these measurements in mind, in August 1963 I.P.E.C. asked Professor N. Hast of Sweden to measure the in situ stresses at the Portage Mountain damsite, the measurements to be incorporated in the design of the powerhouse. The program laid out by Hast was begun in July 1964 and two 40-foot-test holes were soon completed. In August a serious methane explosion in the exploratory tunnel where the measurements were being taken interrupted the program and no further work was done in 1964. An interim report was submitted to I.P.E.C. in September 1964. Work resumed in April 1965 and within two months five more test holes had been completed. A final report was received by June 1965 confirming the suggestion made in the interim report that high horizontal stresses were present in the bedrock.

The magnitude of the maximum horizontal stresses, almost 12 times that expected from the overburden present, was surprising. Well-developed sheeting on the canyon walls west of the powerhouse indicated some stress relief had taken place. Moreover, diametral measurements using an Invar micrometer tape were carried out in the diversion tunnels in 1963, similar to the measurements of Ontario Hydro in the Sir Adam Beck Niagara Tunnels (Hogg, 1959). The results suggested that excavation had not been accompanied by any large rock readjustments. Dolmage and Campbell requested that further rock mechanics studies be undertaken by C-I-M Consultants of Kingston, Ontario under the direction of C.L. Emery. Six oriented samples were collected from the powerhouse area and shipped to the Kingston laboratory for analysis during April and May of 1965. A final report was received in June 1965 which in general confirmed the presence of high residual stresses.

Description and Results of Measurements

Hast Method

The method used by Professor Hast to measure in situ stresses at the damsite is the same as the method he used in Scandinavia (Hast, 1958) and it will be discussed briefly. Using a diamond bit, a 26 mm. diameter hole is drilled in stages to the different depths at which data is collected. At each measuring position a nickel-alloy stress-relieving cell (having magnetostrictive[2] properties) is inserted in the drill hole and oriented in one of the three measuring directions. The cell is loaded to a pre-determined value by wedging it against the side of the borehole. An 87 mm. diameter core is then drilled coaxially with the measuring hole so that the rock composing the core is relieved of its stress, and the original load value of the cell falls. The difference between the original load on the cell and the recorded value after overcoring are then related to the stress in the rock by using the modulus of elasticity for the rock.

A set of three measurements, taken at 60 degrees to each other and lying in a plane perpendicular to the axis of each drill hole, are obtained approximately every three feet over a total length of about 40 feet. The stress ellipse which gives the state of stress in this perpendicular plane is calculated at regular intervals along the drill hole from the three directions of measurement. Determination of the stress ellipsoid, however, which gives the whole state of stress at a point (magnitude and directions of the principal stresses, σ_1, σ_2 and σ_3) requires measurements in three mutually perpendicular drill holes. By drilling two horizontal holes and one vertical hole, nine stress values in three perpendicular planes are obtained from which two ellipsoids may be constructed. One ellipsoid serves as a check on the accuracy of the measurements. The vertical stress measured in the two horizontal holes must be equal, however, in order to carry out these determinations

[2]Magnetostriction is the change in magnetic permeability of a ferromagnetic body as the pressure applied to the body is altered.

(Hast and Nilsson, 1964, p. 604).

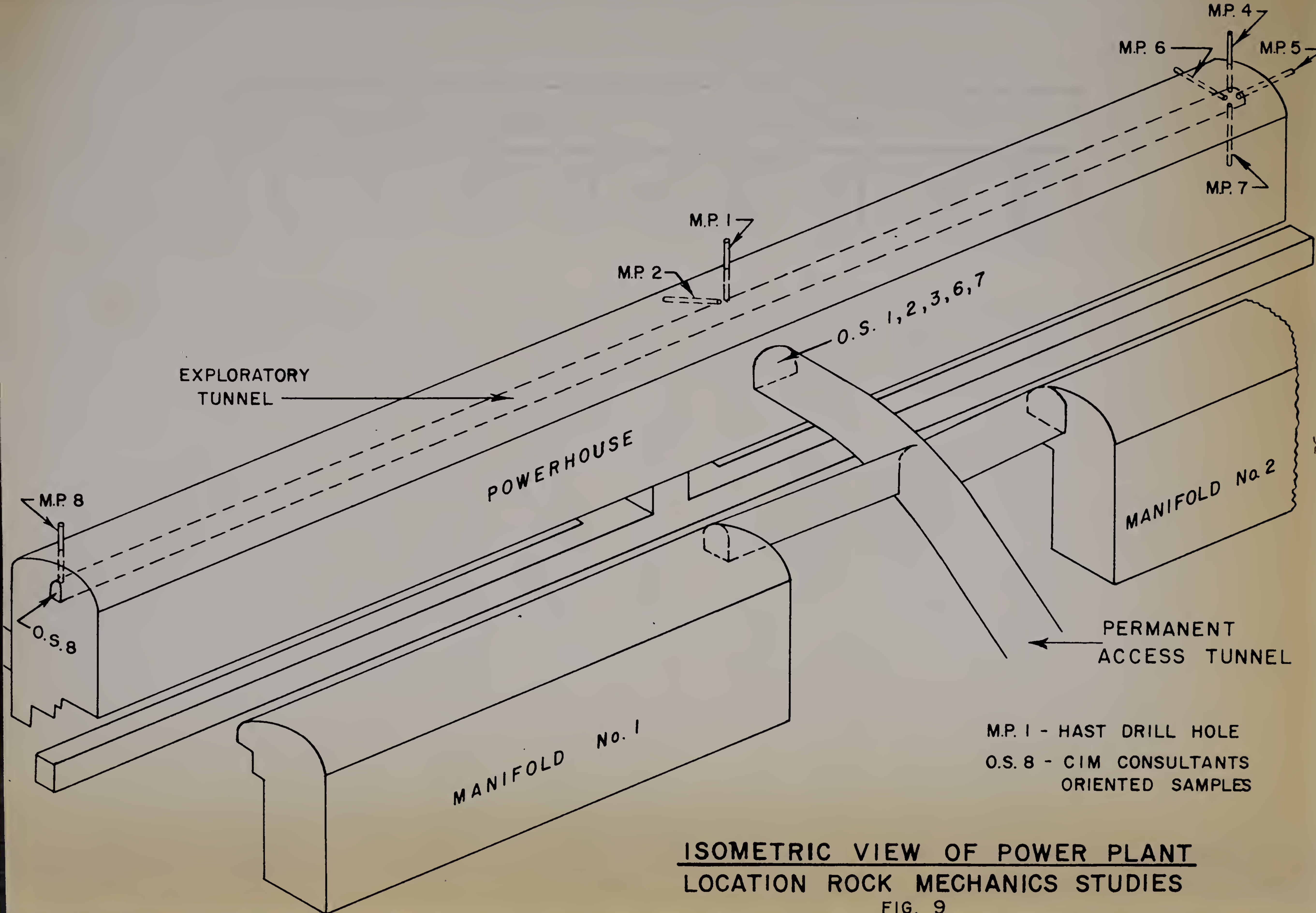
The location and orientation of the seven measuring holes at the Portage Mountain damsite are shown in Figure 9. All holes were drilled from the exploratory tunnel, an eight foot by eight foot pilot drift excavated along the centreline of the proposed powerhouse parallel to the strike of bedding.

Hast Results

From the 307 stress measurements obtained in the seven test holes Hast calculated a total of 48 stress ellipses. Figure 10 (in pocket) shows the results of the ellipse determinations for a typical hole (M.P. 4). No apparent difference exists between the stresses measured in shale and those measured in sandstone. The average magnitude and orientation of the maximum and minimum stresses measured in each drill hole are given in Table 4.

Based on the results of test holes M.P. 1, M.P. 4, and M.P. 7, the maximum horizontal stress in the central and southeastern portions of the powerhouse area has a magnitude of 1920 psi acting in a direction of N 76° E (Fig. 11). The minimum horizontal stress is 1070 psi acting S 14° E. The vertical stress in this region is approximately 1050 psi based on the measurements of M.P. 2 and M.P.5. In the northwest area of the powerhouse the maximum horizontal stress is 1300 psi (N 20° E) and the minimum horizontal stress is 900 psi (S 70° E) (Fig. 11) based on M.P. 8. The vertical stress was not measured at this point.

The orientation of the stress ellipsoid was not determined at any location in the powerhouse region. There were insufficient measurements (see page 30) in the northwest and central areas of the powerhouse. In the southeast area where the required vertical and two horizontal holes were drilled, the vertical stress in M.P. 6 greatly exceeded that measured in M.P. 5.

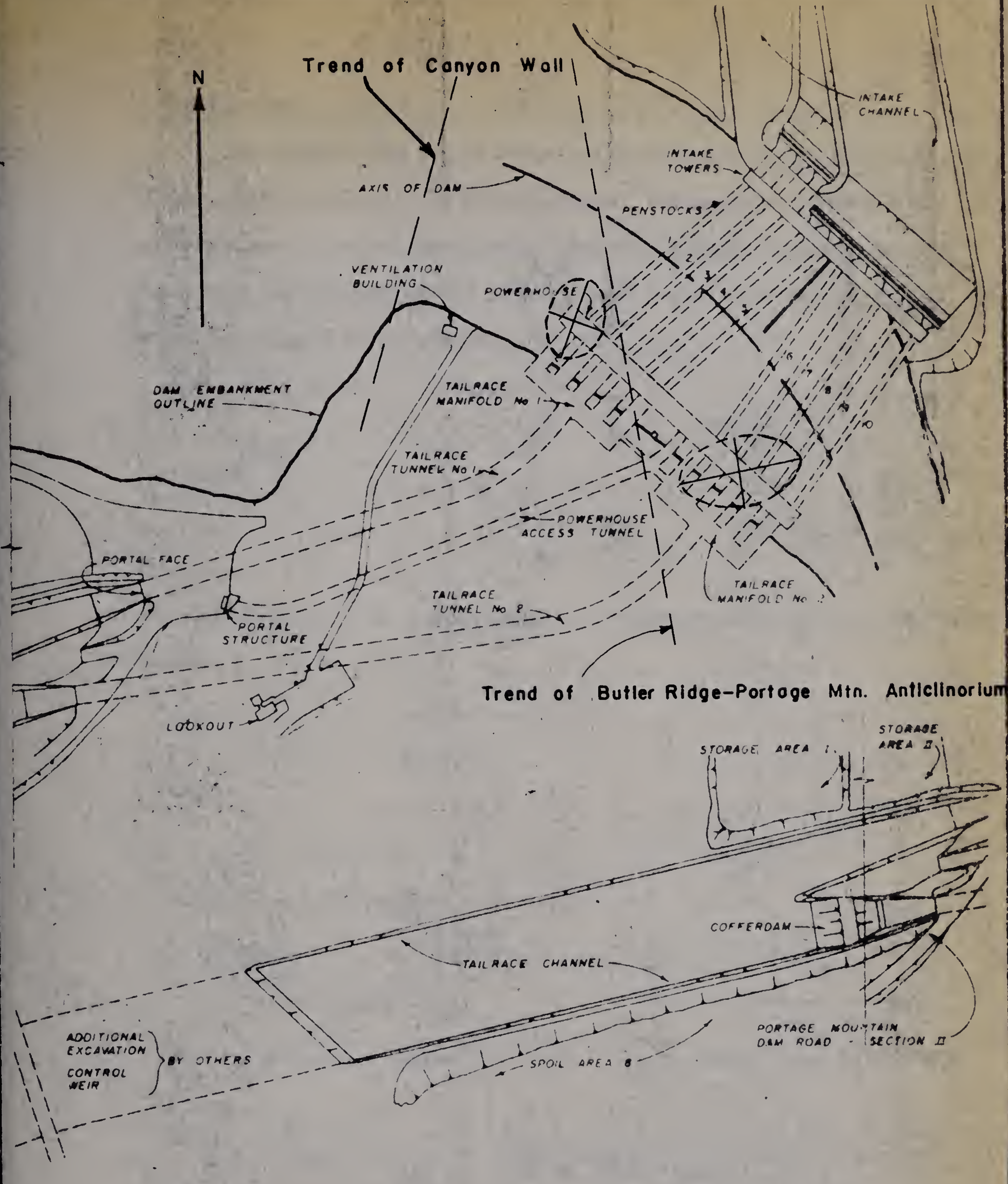


ISOMETRIC VIEW OF POWER PLANT
LOCATION ROCK MECHANICS STUDIES
FIG. 9

TABLE 4

Summary of Hast's Results

	Drill Hole Designation	Drill Hole Orientation	Length (ft.)	Number of Measurements	Average and Maximum Minimum Stresses: Magnitude (psi)	Average and Maximum Minimum Stresses: Direction	Vertical Stress	Remarks
Centre of Powerhouse	M.P. 1	Vertical (up)	42.5	47	$\sigma_1 = 1920$ $\sigma_2 = 1280$	N 70°E S 20° E	-	Horizontal plane of measurement. σ_1 & σ_2 are maximum & minimum horizontal stresses.
Centre of Powerhouse	M.P. 2	Horizontal (due north)	47.6	50	$\sigma_1 = 1300$ $\sigma_2 = 760$	0/40° E 0/50° W	1000	Vertical plane of measurement. Stress ellipse oriented such that long axis dips east at 40°.
Centre of Powerhouse	M.P. 3	Horizontal (due east)	25.8	27	-	-	-	No results as hole incomplete.
East end of Powerhouse	M.P. 4	Vertical (up)	41.	40	$\sigma_1 = 1850$ $\sigma_2 = 925$	N 65°E S 25°E	-	See remarks of M.P. 1.
East end of Powerhouse	M.P. 5	Horizontal (parallel to P/H)	44.	52	$\sigma_1 = 2100$ $\sigma_2 = 1100$	Horizontal Vertical	1100	Vertical plane of measurement. Stress ellipse oriented such that long axis horizontal.
East end of Powerhouse	M.P. 6	Horizontal (perpendicular to P/H)	46.	48	$\sigma_1 = 2060$? $\sigma_2 = 1670$?	43°/60° SE 43°/30° SW	1890?	Vertical plane of measurement. Numerous fractures encountered. Stress measurements in doubt.
East end of Powerhouse	M.P. 7	Vertical (down)	43	27	$\sigma_1 = 2000$ $\sigma_2 = 980$	N 95° E S 05° W	-	See remarks of M.P. 1.
West End	M.P. 8	Vertical (up)	41	43	$\sigma_1 = 1300$ $\sigma_2 = 900$	N 20° E S 70° E	-	See remarks of M.P. 1.



Plan Scale: 1" = 400 ft

Ellipse Scale: 1" = 4000 psi

ORIENTATION of STRESS ELLIPSES, HORIZONTAL PLANE (Host results)

FIG. 11

C-I-M Consultants Method

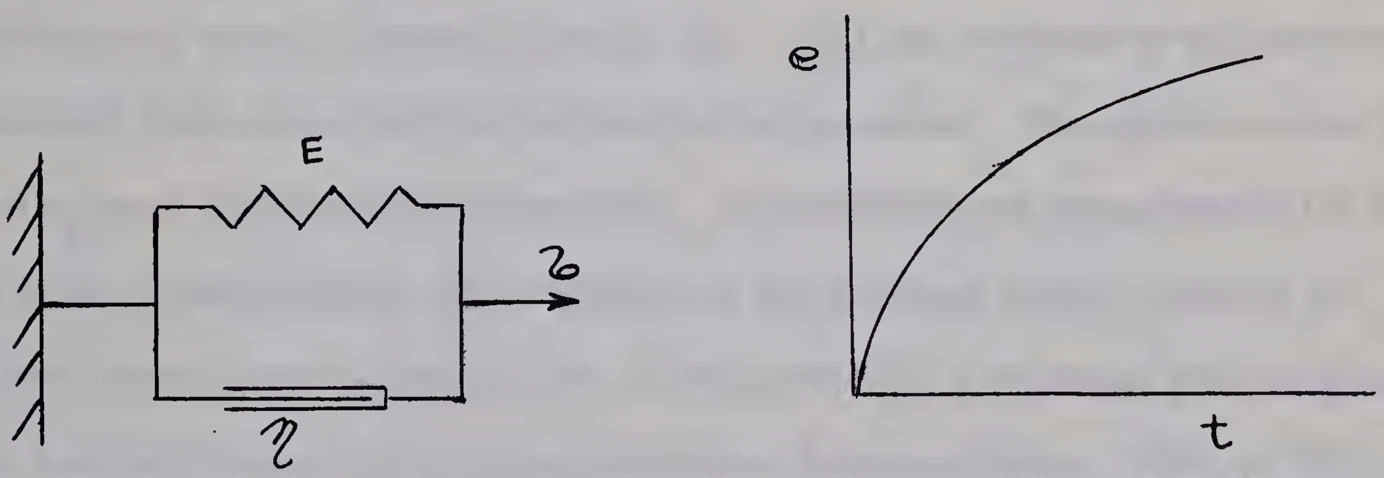
The method used by C-I-M Consultants to analyse the in situ stresses and strains (Emery, 1962) is outlined in the following pages. The basis of this method is that rock contains "conserved elastic strain energy" the value of which depends on its history. Rocks under certain conditions are believed to act as Kelvin Substances (Fig. 12) which are governed by the following equations:

$$\tau = E \cdot e + \eta \frac{de}{dt} \tag{3}$$

from which

$$e = \frac{\tau}{E}\left(1 - e^{-Et/\eta}\right) \tag{4}$$

where τ is unit stress, e is unit strain, E is Young's modulus, η is the coefficient of viscosity, and t is time.



a. Rheological Model   b. Strain-Time Relationship

FIG. 12 KELVIN SUBSTANCE

The rocks are thus assumed to behave as damped elastic bodies although, as Emery points out, in rocks under load maximum strain will only be reached in infinite time. Accordingly, one the load is removed infinite time will be required for return to the

unstrained state. A rock sample removed from its environment, therefore, will attempt to recover from its strained condition, the surface layers of the sample being more able to do so than the inner layers according to Emery.

The six oriented rock samples, collected from the access and exploratory tunnels (Fig. 9), had three inch cubes cut from them. Photo-elastic patches, two inches in diameter, were then placed on three mutually perpendicular faces. These special plastic gauges when strained split incident light into two components vibrating perpendicular to each other and travelling at different speeds within the plastic. When bonded to the oriented rock samples any strain in the rock occurred in the plastic. The directions of the maximum and minimum strains for each face (e_1 and e_2 respectively) were found by polarizing incident light in various directions until the plane of polarizations of the polariscope and the plastic were identical. The magnitude of the differential strain ($e_1 - e_2$), at any point in the plastic, and thus the rock face, is determined from the birefringence produced by the relative retardation of the two components of polarized light in the plastic. The colour, directly proportional to the differential strain, allowed values for ($e_1 - e_2$) to be obtained at any point on the instrumental faces and at anytime during the testing period. The absolute values for e_1 and e_2 were calculated mathematically. By combining the measurements for all three faces of each sample, the orientation of the principal strains, and thus the principal stresses, may be determined. In this method it is assummed that the directions of the principal stresses and principal strains are the same (Emery, 1964, p. 521).

C-I-M Consultants Results

The time-dependent strains were measured for a two to three week period in each sample. The directional properties of the three faces of each sample are shown in Table 5. The maximum strain direction in the horizontal plane averages S 85° E in the access tunnel and S 24° E in the northwest end of the exploratory tunnel.

TABLE 5

Directional Characteristics of Oriented Rock Samples as Determined by C-I-M Consultants

Sample No.	Type	Location	Maximum (e_1) & Minimum (e_2) Strain Directions					
			Horizontal Plane		N-S Vertical Plane		E-W Vertical Plane	
			e_1	e_2	e_1	e_2	e_1	e_2
1	Siltstone	Access Tunnel	095°	005°	32°S	58°N	Vert.	Horiz.
2	Sandy Shale	Access Tunnel	093°	003°	40°N	50°S	60°W	30°E
3	Shale	Access Tunnel	082°	172°	Horiz.	Vert.	15°W	75°E
6	Siltstone	Raise	077°	167°	9°N	81°S	16°E	74°W
7	Sandy Siltstone	Access Tunnel	129°	039°	84°S	6°N	17°W	73°E
8	Siltstone	Exploratory Tunnel	156°	066°	Horiz.	Vert.	4°W	86°E

For sample 2, the directions of e_1 and e_2 for each face were combined to give the orientation of the maximum principal stress (S 85° E/27° E).

The attitudes of the maximum shearing stresses as obtained photoelastically from samples 2, 3, 6 and 8 are N 55° E/11° NW and S 65° E/12° NE. The average magnitudes of the maximum and minimum strains for each face are summarized in Table 6. The magnitudes, which were much higher in the access tunnel than in the exploratory tunnel, were observed to be compressive in all cases.

TABLE 6

Average Time-Dependent Strain Magnitudes as Determined by C-I-M Consultants

Location	Strain	Magnitudes (μ in/in)
	e_1	e_2
Access Tunnel	669	253
Exploratory Tunnel	53	48

Discussion of Results

The Hast and C-I-M methods of stress analysis are based on the theory of elasticity. This theory assumes that rock is uniform, elastic and isotropic. In reality a rock contains many structural or textural features and thus is a heterogeneous, anisotropic mass which cannot be considered a true elastic body. Examples of this anisotropy are seen in the variation in compressive strength and elastic modulus with direction (Table 7). This deviation from the theory of elasticity is well realized today and as noted by Clark (1965), "the greatest current need for research is in the evaluation of effective properties which will define the response of rock masses rather than intact specimens to imposed loads." Until new approaches are outlined, however, the design engineer must use the best information available and

TABLE 7

Summary of Powerhouse Rock Tests, Carried out by Coast Eldridge Laboratory, Vancouver

	Elastic Modulus (10^6 p.s.i.)			Ultimate Compressive Strength (p.s.i.)		
Type of Rock	Along Strike	Along Dip	$\perp$ To Bedding	Along Strike	Along Dip	$\perp$ To Bedding
N4 Sandstone	2.6	2.4	2.3	19,800	19,300	17,700
N5 Shale	5.6	4.4	5.3	13,700	11,800	16,300
N5 Sandstone	2.5	2.2	2.6	16,400	18,800	20,300

this is usually based on elastic theory. As long as allowances are made for theory deviation and for the unforeseen, the results of studies such as those of Hast and C-I-M Constultants may be used as a guide to provide an economic stable structure.

Hast's work suggested that the stresses in the central and eastern portions of the exploratory tunnel (σ_H = 1920 psi, 1070 psi; σ_V = 1050 psi) are much higher than the stresses expected due to gravity alone. Assuming a Poisson's ratio of 0.25, the overburden depth of 450 feet in the powerhouse area would produce a vertical stress of about 500 psi and horizontal stresses of 175 psi.

In analysing Hast's results, some of the assumptions on which any relief method of measuring stress is based (Merrill, 1963, p. 349) are outlined in the following paragraphs.

1. The reduction in stress measured by the nickel alloy gauge as it is overcored is equal to the in situ rock stress.

   Hast's measurements are based on instantaneous elastic recovery strains only. The fact that time-dependent strains have been measured by C-I-M Consultants indicates that the in situ stress is not completely removed at the time of overcoring.

2. Stress calculations assume the rock is elastic, uniform and isotropic.

   That the rock at the damsite is heterogeneous and anisotropic is indicated in Table 7.

3. The elastic constants calculated using samples in the laboratory are the same as the in situ rock.

   The difficulties and dangers in extrapolating laboratory values to the field have been described by Clark (1965).

Despite these sometimes inaccurate assumptions the presence of large horizontal compressive stresses in the bedrock at Portage Mountain damsite seems justified. The magnitudes of these horizontal stresses obtained by Hast is probably conservative as his

method ignores the time-dependent strains. In the central and eastern area of the powerhouse the direction of the maximum horizontal stress (N 76° E) compares favorably with that expected from the regional structure. The trend of the Portage Mountain - Butler Ridge anticlinorium at the damsite is N 10° W (Fig. 6). A regional stress normal to this, would have a bearing of N 80° E. The stress magnitudes in the west end of the powerhouse are lower and the orientations have changed as would be expected due to the proximity of the canyon wall. The maximum horizontal stress trends N 20° E while the canyon wall trends N 16° E (Fig. 11).

The program of C-I-M Consultants was very limited in scope, but whereas the results are somewhat questionable due to the small number of samples, the information is a useful complement to Hast's work. In the horizontal plane the maximum strain or stress axis trends N 95° E in the central powerhouse area and N 24° W in the northwest powerhouse region. This compares with the values of N 76° E and N 20° E as found by Hast's data and is further evidence for a directional change in the stress field as the canyon wall is approached. The attitude of the maximum principal stress was calculated in only one sample giving a value of S 87° E/27° E. Hast's data (Table 4, drill hole M. P. 2) also suggested the maximum principal stress axis plunges east.

The strain magnitudes (Table 6) are of little meaning as the values do not include immediate elastic recovery strain (measured by Hast) or all of the time-dependent strain. No extrapolation was made by C-I-M Consultants to estimate the state of strain when each sample was collected or when the access and exploratory tunnels were excavated. The large difference between sample 8 and samples 1, 2, 3, 6, and 7 is understandable as the exploratory tunnel had been excavated for a year and the access tunnel face had only been exposed two months when the samples were obtained.

In summary, the evidence of Hast and C-I-M Consultants shows that the bedrock at the damsite is subjected to horizontal compressive stresses trending in an approximate east-west direction. The magnitudes of the stresses are unknown but they are larger than stresses resulting from gravitational forces alone.

Plate 2 Powerhouse Arch Looking Southeast at Completion of Excavation

DESIGN OF THE POWERHOUSE ARCH

Introduction

The basic design of the underground powerhouse and ancillary works was completed in May 1965 by I.P.E.C. in Vancouver with the assistance of their field engineers and consultants. This section describes some of the major considerations that went into (1) choosing the location and orientation of the powerhouse, (2) designing the shape of the roof, (3) requiring that stress-relieving be carried out during excavation, and (4) deciding the nature of the support of the roof.

A basic support design must be kept flexible in order to deal with unforeseeable field conditions. A body of rock is rarely homogeneous, and certain structural discontinuities, potential planes of weakness, may require localized modifications of the basic support. In order to test the specified support at the Portage Mountain damsite and to try to anticipate possible field problems, a preliminary rock bolt testing program was conducted prior to excavation of the powerhouse. The results of this program carried out under the direction of the I.P.E.C. Geology Department and a contractor's representative, are discussed in this chapter.

Basic Design Theory

Location of Powerhouse

In the preliminary design the powerplant was a concrete structure at the base of the plateau, downstream from the right abutment. However, after experience with the diversion tunnels had suggested that large excavations would stand without an uneconomic amount of support, the powerplant was changed to an underground structure beneath the left abutment. The main factors controlling the final location of the powerplant were the cost of the penstocks and the position of the overlying embankment. In one of the initial layouts the penstocks were almost vertical to make them as short as possible. This required that the powerhouse cavern underlie the

central part of the dam which is undesirable due to the added weight of overburden above the arch. In the final design, the powerhouse was placed directly below the downstream toe of the dam (Fig. 5).

Geological factors were not an important consideration in deciding the location of the powerhouse. The long axis of the chamber, however, was oriented along the strike of bedding which subparalleled the axis of the dam on the left abutment (Fig. 7).

Shape of Powerhouse Roof

A semi-elliptical arch (Fig. 13) was chosen for the roof of the powerhouse as this cross-section theoretically provides satisfactory stress distribution around the opening. At the time the powerhouse design was finalized, limited data was available from the Hast program to help predict the magnitude of the stresses that would be present above the arch.

Little consideration was given to the prospect of serious overbreak occurring. The contractor was required to excavate to the "A" line although excavation was paid for to the "B" line (Fig. 13). A penalty was imposed for excavation or overbreak beyond this "B" line.

Stress Relieving During Excavation

The method of excavation was left to the discretion of the contractor with the specification that after excavation of the arch and prior to its concreting, a 34-foot bench immediately below the arch had to be blasted and partially excavated. The blasting was to allow wall rock relaxation and inward wall movement to occur before concrete emplacement. This stress relieving process was especially critical in the event the final stress measurements of Hast confirmed his high initial results.

Support of Powerhouse Arch

Experience gained from the excavation of the three diversion tunnels assisted the design engineers in laying out a rock bolt pattern for the powerhouse

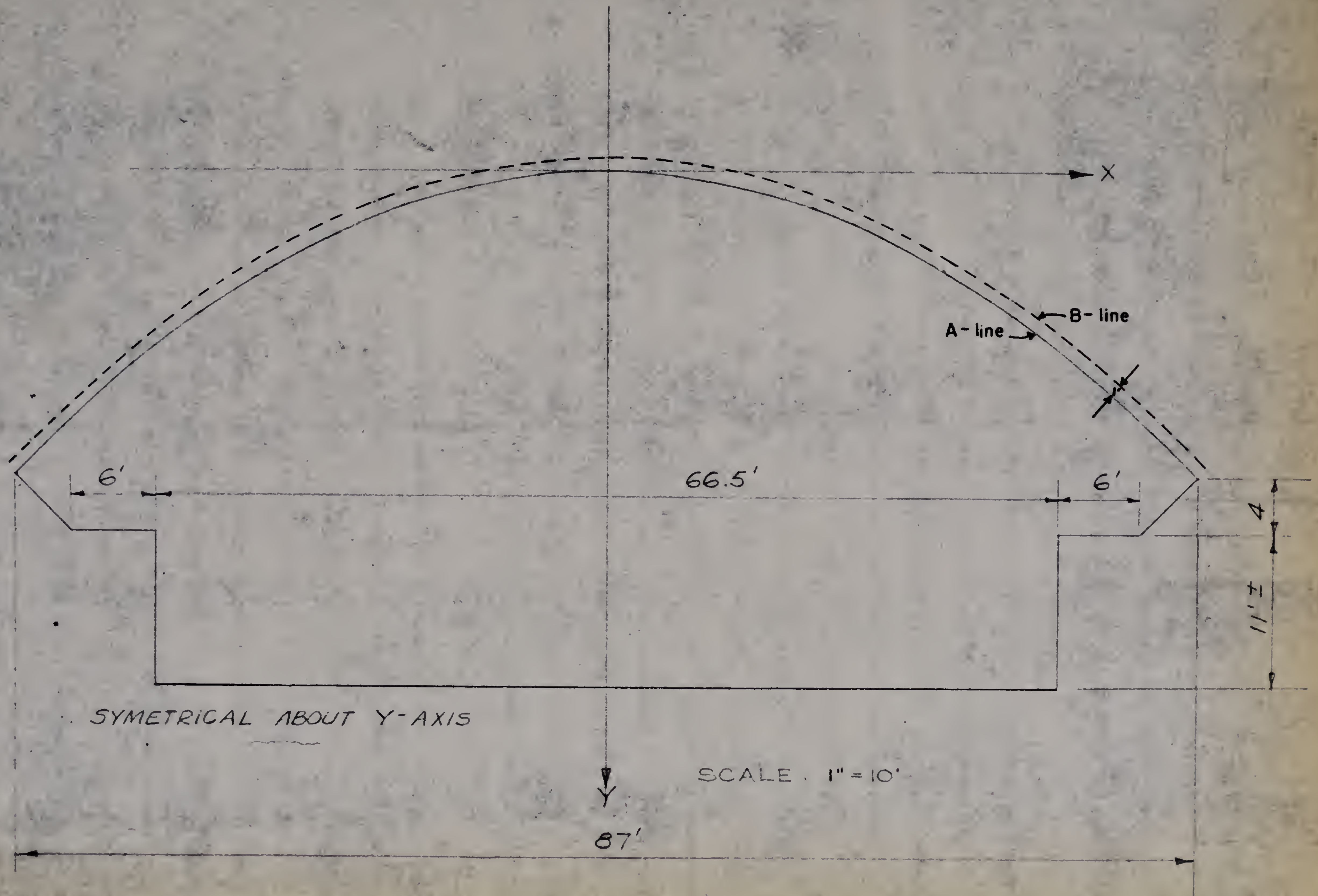


FIG. 13 CROSS-SECTION of POWERHOUSE ARCH

arch. In the diversion tunnels 10-foot-long tensile steel rock bolts spaced five feet apart had been specified in only those areas where shale formed the arch of the tunnels. During actual excavation it became apparent that rock bolts six to eight feet long would be sufficient except where anchorage in sandstone was desirable. No rock failures occurred in the diversion tunnels.

The width of the powerhouse exceeded the diversion tunnel span by almost 40 feet, but the shale forming the powerhouse arch was the same as encountered in the tunnels. High tensile steel bolts (ASTM A-306 grade 80, one-inch-diameter) with minimum lengths of eight to ten feet and five feet apart were considered necessary. Geological cross-sections of the arch rock suggested that the best anchorage would be provided approximately 14 feet above the "A" line and the minimum length of bolt was therefore increased to 14 feet. This pattern of fanned 14-foot-long bolts, at five foot spacing, in addition to arching from the abutments, was judged to be adequate for the basic support of the powerhouse roof. The N5 Shale - N4 Sandstone contact, approximately 16 feet above the top of the arch, was known to be a potential parting plane and the length of the central five bolts was increased to 20 feet so as to anchor in sandstone and increase the friction across this plane (Fig. 8).

The contract specifications called for the pattern rock bolts to be installed within five feet of the working face within eight hours after blasting. These bolts were to be tensioned to two-thirds of the yield strength of the rock bolt at the time of installation. If the rock was found to be incapable of developing the specified anchorage, allowance was made for a reduction in spacing of the rock bolts and a corresponding reduction in the anchorage requirements to suit the capability of the rock. All bolts within 30 feet of a working face had to have the torques checked after each blast and retightened if necessary. Wire mesh and steel strapping were not specified throughout the arch but were to be used as conditions

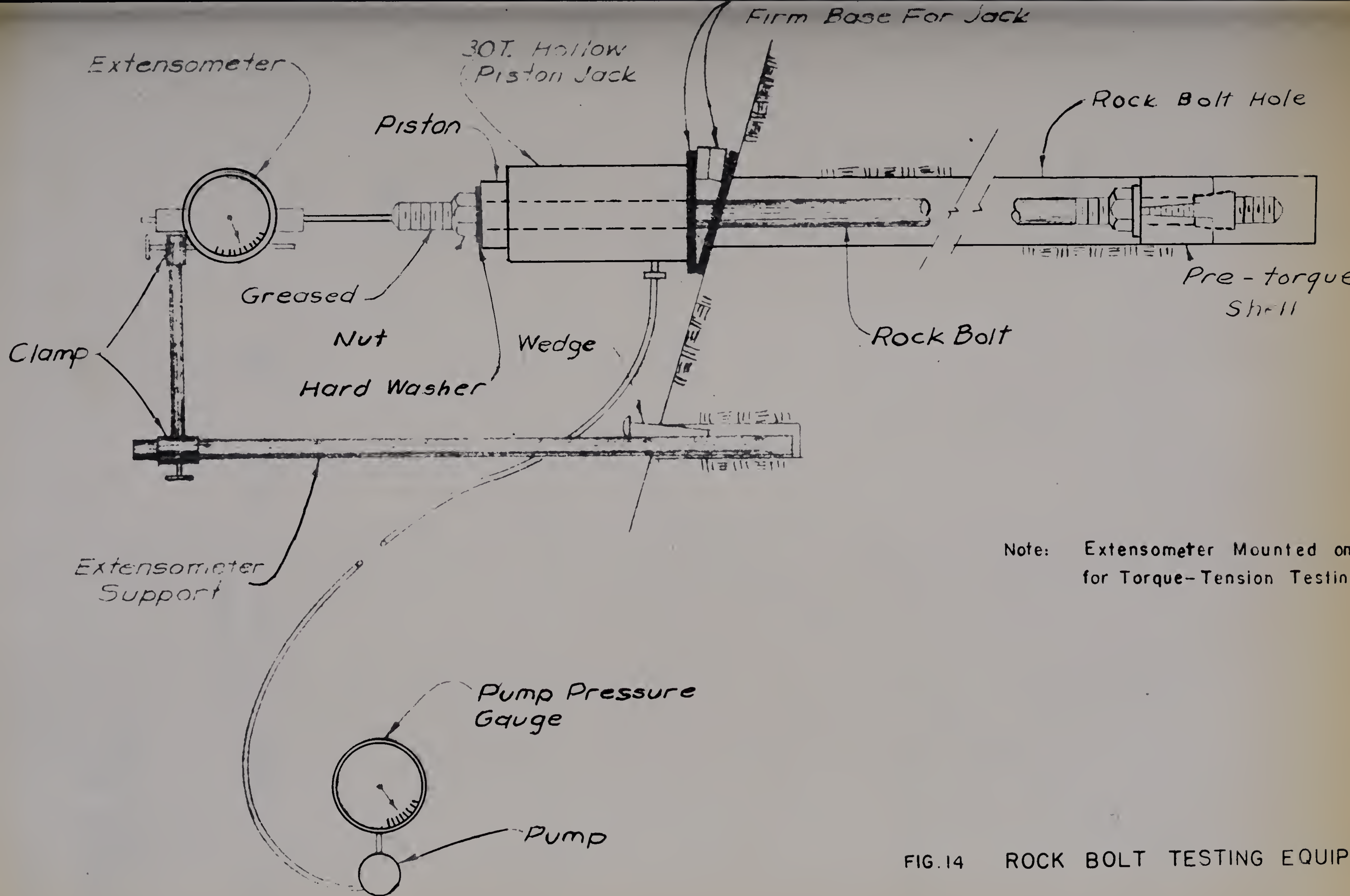
required. Finally all pattern bolts had to be grouted for permanent support, although no grouting was permitted within 100 feet horizontally of an excavation face. After stress relieving (page 43), a thin, reinforced concrete arch was then to be emplaced to assist with the permanent support of the arch.

Field Testing

As the final phase of the support design prior to actual excavation of the powerhouse, the rock bolt selected by the contractor was tested in the wall and back of the permanent access tunnel where the lithologies are the same as those of the powerhouse arch. These tests were carried out to establish a torque-tension relation for the rock bolt and to establish whether the anchorage assemblies would develop and maintain two-thirds of the yield strength of the rock bolt.

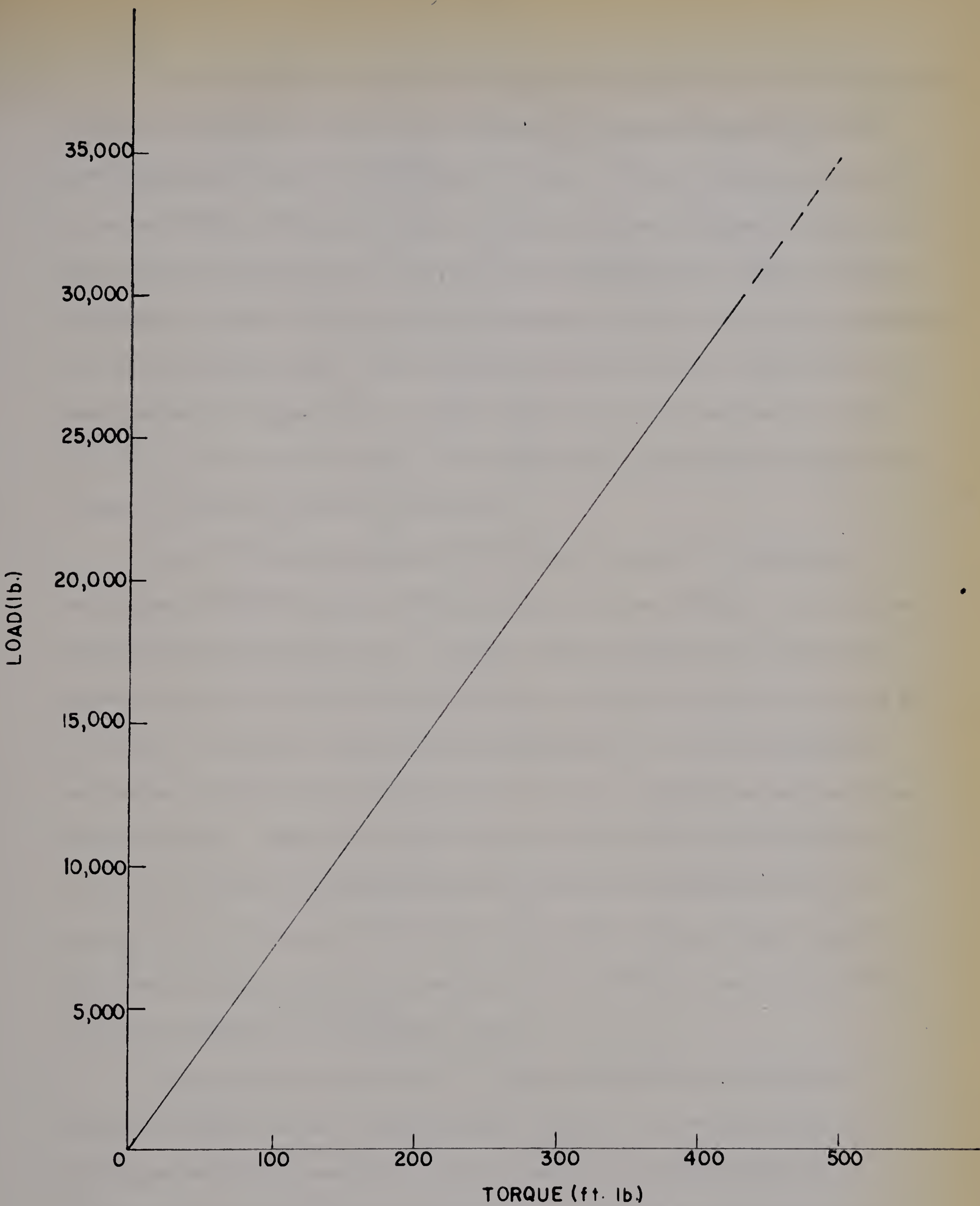
The contractor selected a one-inch-diameter, high-tensile, steel rock bolt with a central 1/4 inch diameter grout hole running throughout its length. From manufacturer's data the yield strength of the bolt is 35,000 pounds and the ultimate strength 50,000 pounds. In order to determine a torque-tension relation 17 rock bolts were installed horizontally in sandstone in the access tunnel. A test jack was placed on the bolt between two washers one of which rested directly on the rock surface (Fig. 14). After a hard washer and nut were placed on the outer end of the bolt, the nut was torqued in 25 foot pound increments from 75 to 400 foot pounds; the gauge reading on the pump (the bolt tension) was then recorded. An extensometer was used but it was mounted on the jack rather than on the end of the bolt so that the extensometer readings were practically meaningless in the initial torque ranges.

The results shown in Figure 15 indicated that a torque of 325 foot pounds on the nut was required to impose a tension of 23,000 pounds or two-thirds of the yield strength. By extrapolation a torque of 500 foot pounds corresponds to the yield strength of 35,000 pounds under ideal conditions.



Note: Extensometer Mounted on Jack for Torque-Tension Testing

FIG. 14 ROCK BOLT TESTING EQUIPMENT



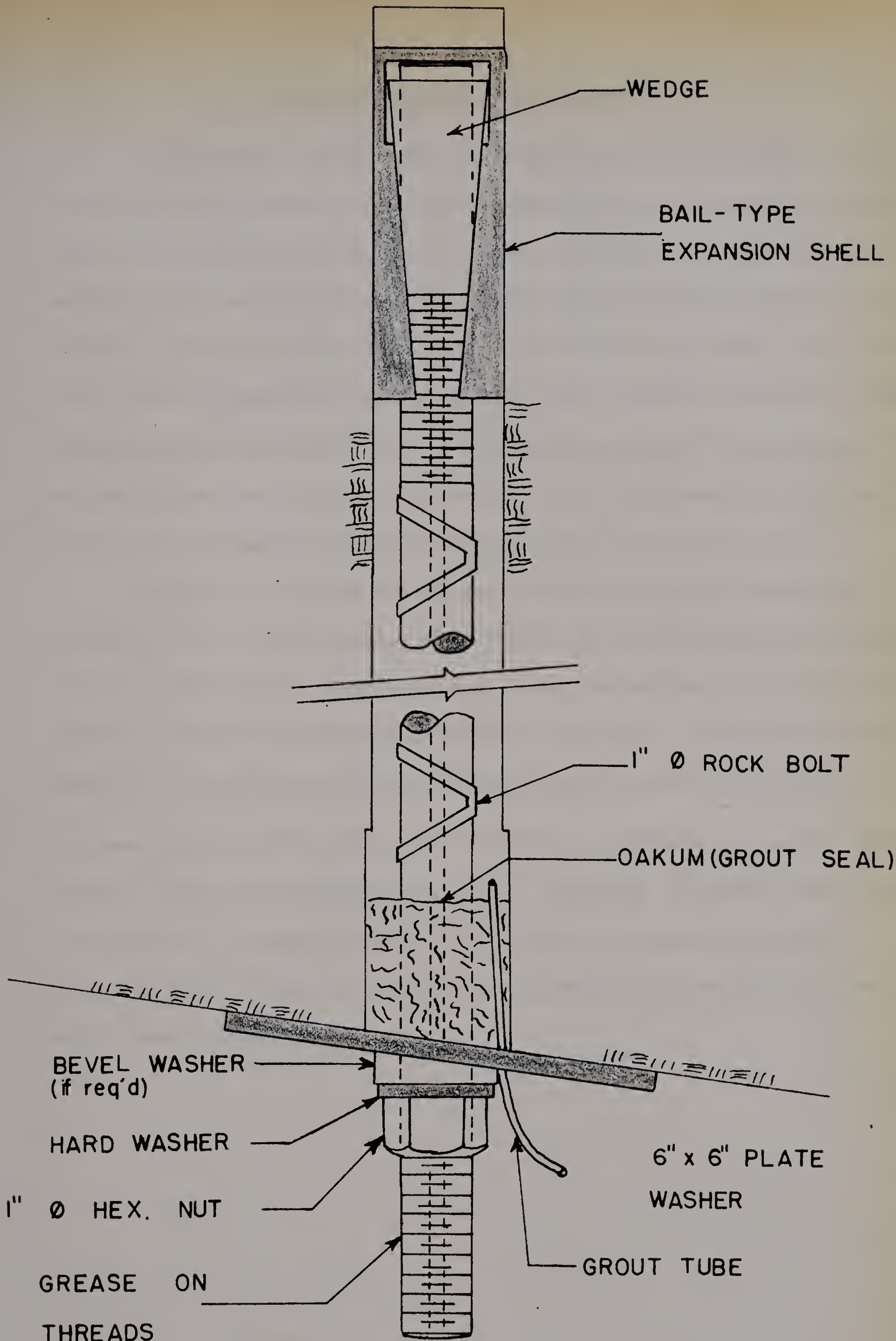
TORQUE-TENSION RELATION
1" Ø ROCK BOLT

FIG. 15

The second phase of the testing involved anchorage pull tests in both shale and sandstone to determine whether the two different anchorage assemblies selected would develop and maintain 23,000 pounds tension. The pull tests were conducted at various angles to the strata using the arrangement shown in Figure 14. Additional pull tests were carried out on installed or tensioned bolts using a slightly modified arrangement. Both of the two anchorage assemblies or shells chosen by the contractor were of the expansion type. The first was a pre-torque variety in which the anchorage is set before tension is put on the bolt (Fig. 14). The second shell was a bail type (Fig. 16) in which the anchor is set and the bolt is tensioned at the same time. A total of 45 pull out tests was conducted.

Results of these tests showed that there was no problem in obtaining an anchorage of 35,000 pounds in sandstone using either type of shell. This was expected from the torque-tension tests. In shale a rated anchorage of two-thirds of the yield strength could not be consistently obtained with bolts oriented in the range of 0° to 30° to the strata. With bolts inclined 45° and 90° to strata the specified anchorage could be obtained although a maximum value (32,000) pounds was below that of sandstone. Again the strength of the anchorage did not vary according to the type of shell used but a rather high amount of pre-torque (300 foot pounds) was necessary for the first type of shell when used in shale. The contractor chose the bail type of shell for use in the powerhouse as the pre-torque type was very difficult to set even with the use of an impact wrench.

Eleven bolts were installed in the weakest shale of the access tunnel to determine whether the bolts would be able to maintain their tension in the rock types expected in the powerhouse arch. All bolts were initially installed with 23,000 pounds tension. Records were kept over a six week period during which time no significant torque losses were noted in any of the bolts.



1" Ø HOLLOW ROCK
BOLT ASSEMBLY

FIG. 16

Summary of Final Rock Bolt Design

The preliminary rock bolt testing program suggested that the support pattern developed for the powerhouse arch would be satisfactory as a basic design. All of the test bolts anchored in sandstone and the majority (70 per cent) of the test bolts anchored in the weakest shale were able to develop the required 23,000 pound load. However, the number of 20-foot-long bolts was increased to nine (Fig. 17) so that as many bolts as possible would be anchored in either sandstone or siltstone. Although the specifications did allow for bolting in addition to the basic five foot pattern, this extra bolting was expected to be provided only in those areas where the basic pattern bolts were not able to develop the required 23,000 pounds tension.

No definite conclusion was reached as to the best angle of inclination of the rock bolts to the strata because of the limited data available and the fan arrangement as originally laid out was maintained. It was realised that rock bolting must take into consideration the planes of weakness in the rock. If this is not done the possibility of shearing along these planes could actually increase (Lang, 1957, p. 18). However, jointing in the vicinity of the arch was not considered well enough developed to modify the orientation of the bolts. In addition, the planes of maximum shearing stresses as measured by C-I-M Consultants were shown to be dipping gently (10° to 12°) so that the rock bolts would generally be installed at a large angle to these planes.

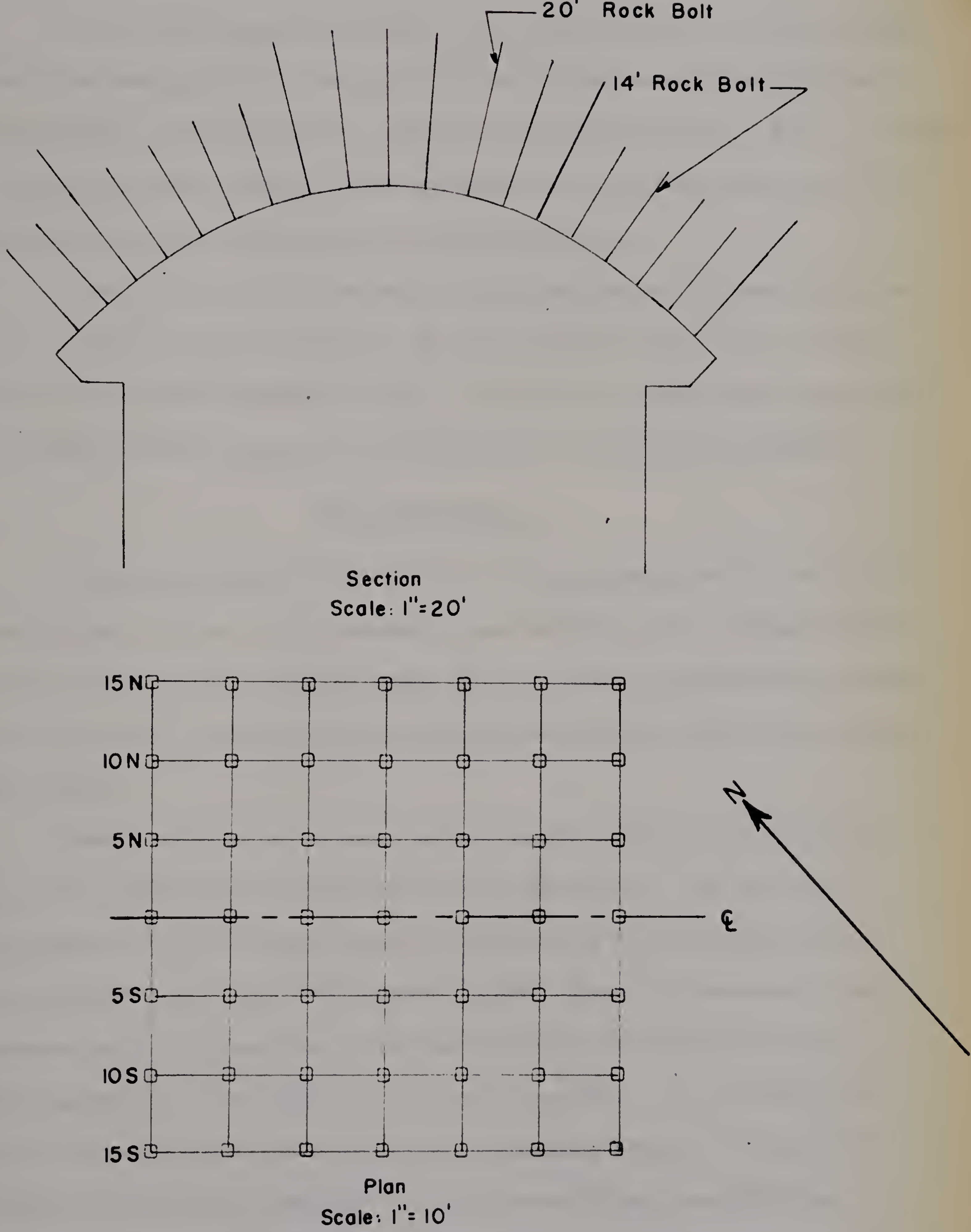


FIG. 17 POWERHOUSE ARCH

5' x 5' BASIC ROCK BOLT PATTERN

DEFORMATION OF POWERHOUSE ARCH

Introduction

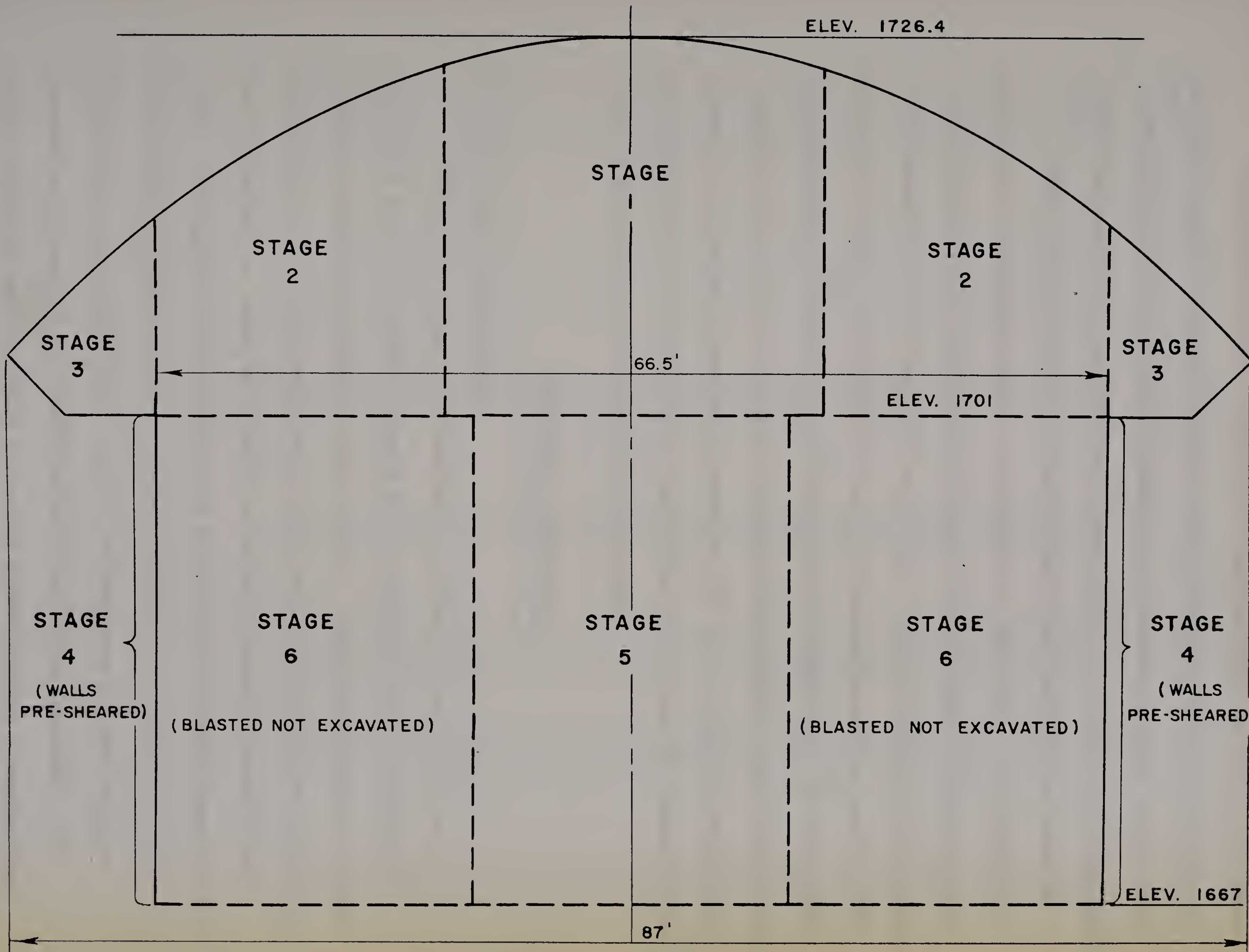
In the initial stages of excavation the contractor became concerned with the steadily increasing load on a number of installed rock bolts, as indicated by torque measurements, and questioned the stability of the powerhouse arch. I.P.E.C. decided to observe the amount and rate of rock movements using additional methods so as to provide enough data to determine the likelihood of failure.

A brief account of the contractor's excavation sequence is given as this has some bearing on the rock movements. The various methods used to carry out these observations are then described. Finally, a résumé of the supplementary support that was required and some opinions on the nature of the movement are presented.

Excavation Sequence

The contractor began the excavation of the powerhouse arch by ramping up from the previously excavated permanent access tunnel (Fig. 9). This ramp entered near the top of the arch in approximately the centre of the powerhouse and provided two working faces so that excavation could proceed simultaneously to both ends of the chamber.

After completion of the ramp, central headings, 26 feet wide and 25 feet high, were driven towards the extremities of the powerhouse. This first stage of excavation was to be followed closely by excavation of two side faces, 20 feet wide and two haunch faces 10 feet wide (Fig. 18). The mining sequence in the northwest end of the powerhouse proceeded uniformly with the side and haunch faces generally in close proximity to the central heading. In the southeast end of the chamber excavation did not advance in the same manner, for the central heading was driven well ahead of the side and haunch faces, especially those on the southern side of the powerhouse. Figure 19 (in pocket) shows the daily and



POWERHOUSE EXCAVATION SEQUENCE
ELEV. 1726 – 1667
FIG. 18

Scale: 1" = 10'

monthly progress of the arch excavation.

Installation of pattern rock bolts followed blasting of each face. Supplementary bolts required in certain areas were either installed at the time of excavation or later depending on information available from instrument readings. In the initial stages of excavation only very limited use was made of wire mesh and steel strapping.

After excavation of the arch and grouting of the rock bolts the contractor pre-sheared the powerhouse walls to an elevation of 1667 feet. This pre-shearing was accomplished by drilling three-inch-diameter vertical holes, 34 feet long at one foot intervals for the entire length of the chamber. Alternate holes were then blasted. Removal of a 22-foot-wide slot in the centre of the powerhouse from elevation 1701 feet to elevation 1667 feet for the full length of the chamber (Fig. 18) was the fifth stage of the excavation. In stage six the side benches on either side of this slot were blasted so that the muck expanded to elevation 1701 feet. Following concreting of the arch at this time and subsequent removal of the broken rock, the downward excavation of the main body of the powerhouse proceeded.

Deformation Measurements

Rock Bolt Torque Measurements

Checking the torque of all previously installed bolts within 30 feet of an excavation face was specified to ensure that no reduction in the installed torque (325 foot pounds) occurred due to nearby blasting. Using standard torque wrenches the contractor, in making some of these checks early in the excavation, observed that the torque on a number of the bolts was actually increasing above 500 foot pounds. This suggested the bolt loads were approaching the yielding range of the steel (35,000 - 37,000 pounds). A number of testing areas consisting of about 35 bolts each, were established in the arch and these were checked at regular intervals to try to gain a bolt loading pattern as excavation proceeded. The results of one such test area (Table 8) show the gradual increase in loading after

TABLE 8

Rock Bolt Torque Measurements (foot - pounds)

Location	Date Installed	Installed Reading	11/1/66	19/1/66	28/1/66	7/2/66
363-12N	23/12/65	325	370	325	385	260
363-8N	22/12/65	325	470	440	500	480
363-4N	22/12/65	325	350	280	310	320
363-O	22/12/65	325	360	420	465	620
363-4S	22/12/65	325	360	-	455	555
363-8S	22/12/65	325	315	325	390	460
364-12S	23/12/65	325	360	350	430	520
368-12S	23/12/65	325	385	420	490	600
367-8S	22/12/65	325	330	350	450	520
367-4S	22/12/65	325	340	340	445	505
367-O	22/12/65	325	465	480	565	625
367-4N	22/12/65	325	380	345	425	470
367-8N	22/12/65	325	265	260	320	350
367-12N	23/12/65	325	290	340	450	480
371-9N	21/12/65	325	300	250	125	140
371-4N	21/12/65	325	300	340	410	470
371-O	21/12/65	325	300	340	370	505
371-4S	21/12/65	325	330	325	455	510
371-8S	21/12/65	325	320	340	390	415
371-12S	21/12/65	325	320	210	365	420
375-12S	21/12/65	325	370	425	515	560
375-8S	21/12/65	325	300	280	340	400
375-4S	21/12/65	325	240	280	500	570
375-O	21/12/65	325	325	335	455	495
375-4N	21/12/65	325	300	380	445	440
375-8N	21/12/65	325	280	380	405	500
375-12N	23/12/65	325	280	300	340	415
379-12N	21/12/65	325	380	440	500	540
379-8N	18/12/65	325	360	400	475	540
379-4N	18/12/65	325	400	440	540	560
379-O	18/12/65	325	440	490	530	640
379-4S	18/12/65	325	460	525	-	-
379-8S	18/12/65	325	370	425	545	600
379-12S	21/12/65	325	360	445	555	650
371-12N	23/12/65	325	360	390	430	Bolt out
Average Reading		325	345	368	435	475

The location of a bolt is designated by the distance in feet from the north west end of the powerhouse (e.g. 363 ft.), and the distance in feet north (N) or south (S) of the centre-line of the arch (e.g. 4N).

installation.

The measured torque on a rock bolt is at best only an approximate indication of the load even though a torque-tension relationship had been determined. This is because an increase in the torque necessary to turn the nut can be caused by (1) galling of the threads on the end of the rock bolt, (2) the absence of lubricant on the threads, (3) the rock bolt not being perpendicular to the plate washer, and (4) the presence of a 'soft' steel washer between the nut and the plate washer, as well as by an increase in the load on the bolt. At the damsite test results showed that if a molybdenum base lubricant and a hardened steel washer (ASTM 325 specification) were used on a bolt, the torque required to provide a load of 23,000 pounds on a bolt, was 325 foot pounds. Without the lubricant and hard washer the torque required for the same load was 410 foot pounds. Because of these influences on bolt torque, other methods were used to confirm the existence of high bolt loadings.

Load Indicators

The second method of obtaining the tension in the rock bolts made use of commercially available tension indicators. These circular pads, approximately five inches in diameter, have a 3/8 inch thickness of rubber bonded between two 3/8 inch sections of steel. The rubber is free to expand in diameter as increasing load is put on the steel. These indicators are used in place of a plate washer in a rock bolt assembly and the load on a bolt is measured by a specially calibrated circular gauge.

The indicators were effective in the low loading ranges ($<$ 20,000 pounds) but increases in bolt tension above this range (determined by pull tests) made very little difference in the expansion of the rubber so these devices were discarded.

Rock Bolt Pull Tests

Anchorage pull testing was the main method used to confirm or disprove the existence of a high load on a bolt showing a high torque. With the same equipment which had been used in the preliminary testing (Fig. 14), the pull on the end of the bolt was increased to the point where the hard steel washer could be moved by light tapping with a hammer. The pull at the point was assumed to be the load on the bolt, although the true load may be slightly less (American Mining Congress, 1959). The results of these tests are given in Table 9. Tests 10 and 16 are examples of high torques (700 foot pounds) which were caused by damaged threads. Generally, however, the results of the pull tests demonstrated that a torque in excess of 500 foot pounds indicated a bolt loading of 35,000 to 37,000 pounds, or the yielding range of the steel. With few exceptions the maximum load measured on any rock bolt did not exceed this range.

On the basis of the initial pull tests results the contractor believed that most of the pattern bolts were being subjected to excessive loading, and changed the basic rock bolt pattern to a four-foot spacing in December 1965.

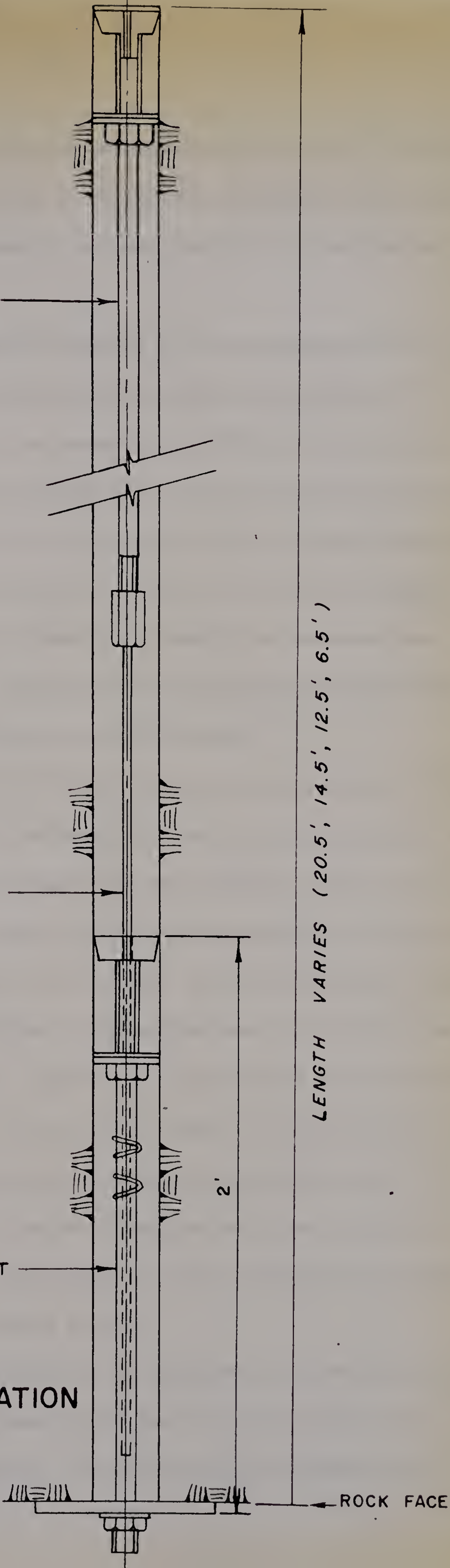
Rock Relaxation Gauges

When it became apparent from the pull tests that a high percentage of the initially installed rock bolts were being increasingly loaded due to the downward movement of the arch, it was decided to measure these movements quantitatively using rock relaxation gauges. The main portion of each gauge consisted of a steel rod 4 1/2 feet long and 3/16 inches in diameter, coupled to a standard 16 foot long, 3/4 inch diameter rock bolt (Fig. 20). This main portion of the gauge was anchored near the base of a vertical 22 foot drill hole using a special hollow wrench. At the collar of the hole a two foot long, one inch diameter hollow rock bolt was anchored in the surface rock. The outer end of the 3/16 inch steel rod rested freely within the hollow rock bolt. The distance between the machined end

TABLE 9

Powerhouse Arch Rock Bolt Pull Tests

Test No.	Test Location Rock Bolt No.	Test Date	Torque Before Test	Load On Bolt	Torque After Test	Remarks
1	477-15S	15/11/65		30,800±	Not checked	
2	479-O	17/11/65		30,800±	"	
3	479-O	18/12/65	820	37,800	"	
4	439-5N	18/12/65	620	37,800	400	
5	444-O	18/12/65	210	14,00	120	
6	410-5N	12/1/66	570	32,200	330	
7	552-O	12/1/66	330	23,800	200	
8	425-O	12/1/66	380	36,400	380	
9	363-4S	13/1/66	370	23,100	240	
10	468-9S	10/2166	1200	28,000	970	Damaged threads
11	467-9N	10/2/66	720	35,000	480	
12	434-6S	11/2166	830	36,400	660	
13	495-10N	13/2/66	850±	34,300+	790±	Washer did not loosen
14	495-5N	13/2/66	750±	37,800+	Not checked	"
15	474-5S	13/2/66	700±	37,800+	500±	"
16	472-10S	13/2/66	750±	28,000	520±	Damaged threads
17	470-5N	13/2/66	600±	35,000	460±	
18	379-12S	18/2/66	690	33,600±	500	
19	379-8S	18/2/66	610±	34,300	420	
20	401-)	19/2/66	940	39,200	460	
21	397-3N	19/2/66	500	37,800	450	
22	344-12S	22/2/66	600	35,000	370	
23	379-O	27/2/66	800	38,500	370	
24	592-12N	27/2/66	420	18,900	320	
25	379-4N	27/2/66	575	36,400	500	
26	290-O	27/2/66	590	32,900	375	
27	688-8N	18/3/66	835	39,200	805	
28	700-8N	18/3/66	740	37,800	665	
29	70-8S	18/3/66	860	30,800	680	
30	716-4S	18-3/66	605	39,200	485	
31	720-4N	18/3/66	725	40,600	630	



INSTALLED ROCK RELAXATION GAUGE

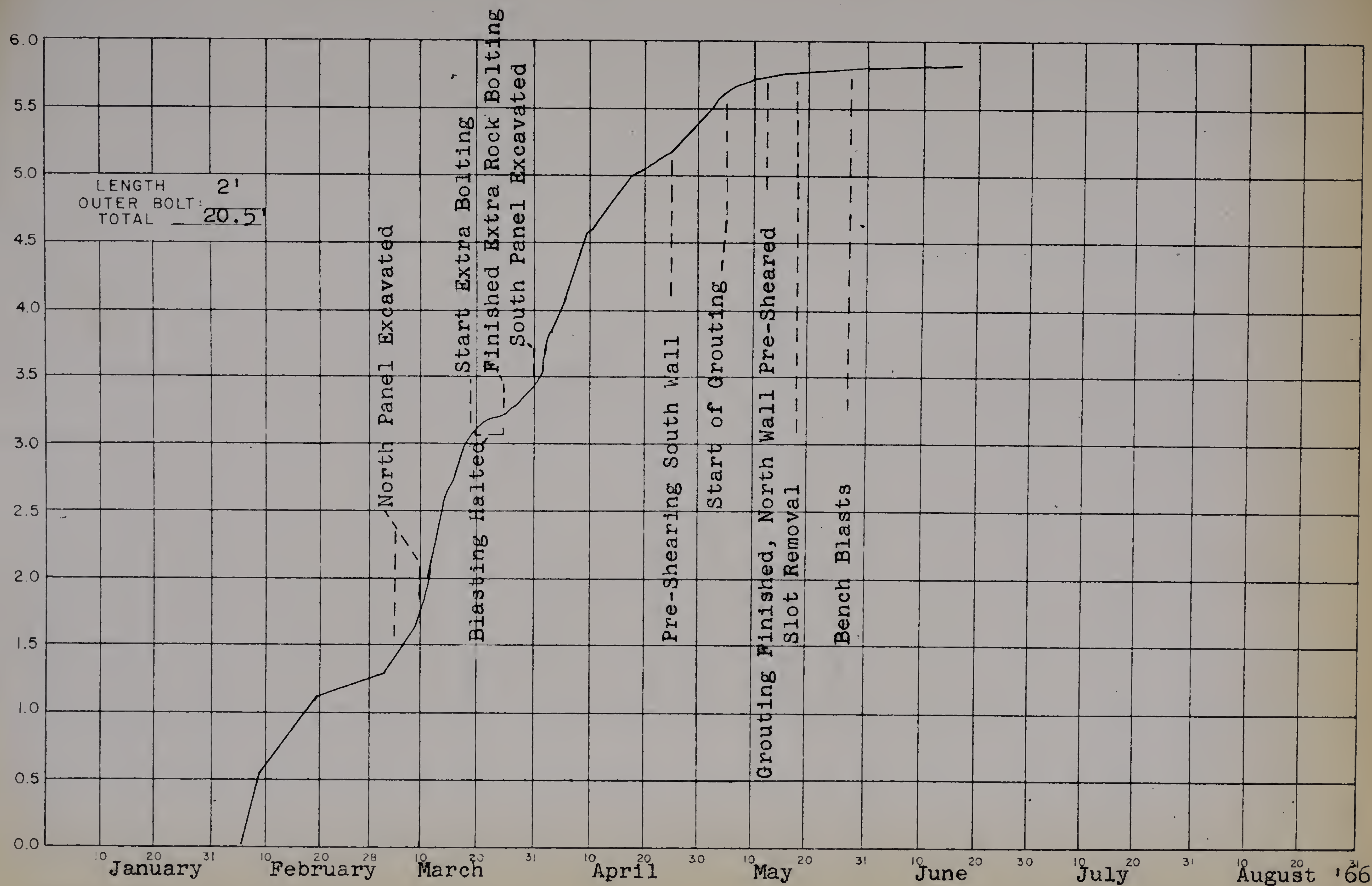
FIG. 20

of the rock bolt and the end of the steel rod was measured at the time of installation and at subsequent intervals. An increase in this distance indicated a relative downward movement of the strata lying between the outer two foot rock bolt and the upper end of the gauge.

Gauges were installed along the centreline of the powerhouse arch every 20 to 30 feet. The standard gauge was 20.5 feet long so that it was anchored in N4 Sandstone just above the contact with the underlying N5 Shale. The movement measured below this upper anchor approximates the total downward movement in the shale which formed the immediate arch if the relaxation of the massive N4 Sandstone is assumed to be very small. At five locations in the arch a set of three gauges of different lengths was installed to try to determine if most of the movement was restricted to a particular horizon. Gauges were also installed at the sides of the arch north and south of the centreline but on a wider spacing.

A total of 70 gauges (Fig. 19) was installed between December 28, 1965 and June 1, 1966. The gauges were all installed at the time of arch excavation at each location except in the vicinity of the powerhouse ramp (station 3 + 65 to station 5 + 50). The arch movement in the region northwest of the powerhouse ramp was far less than that recorded in the region to the southeast. In the latter region, movements along the centreline averaged 2.5 inches reaching a maximum of 5.8 inches near station 7 + 10, while to the northwest, the movements averaged 0.40 inches reaching a maximum of about one inch in the vicinity of station 3 + 50 to station 4 + 50. The movement recorded by each side gauge was less than the adjacent centreline gauge but the pattern of movement is similar for each region of the powerhouse. There is no conclusive evidence to suggest that there was a differential movement between the northern and southern sides of the arch.

Displacement - time graphs plotted for the important gauge sets at stations 7 + 10 and 8 + 05 reveal some significant factors about the nature of the arch movements (Fig. 21 to Fig. 26 inclusive). The arch movements appear to be

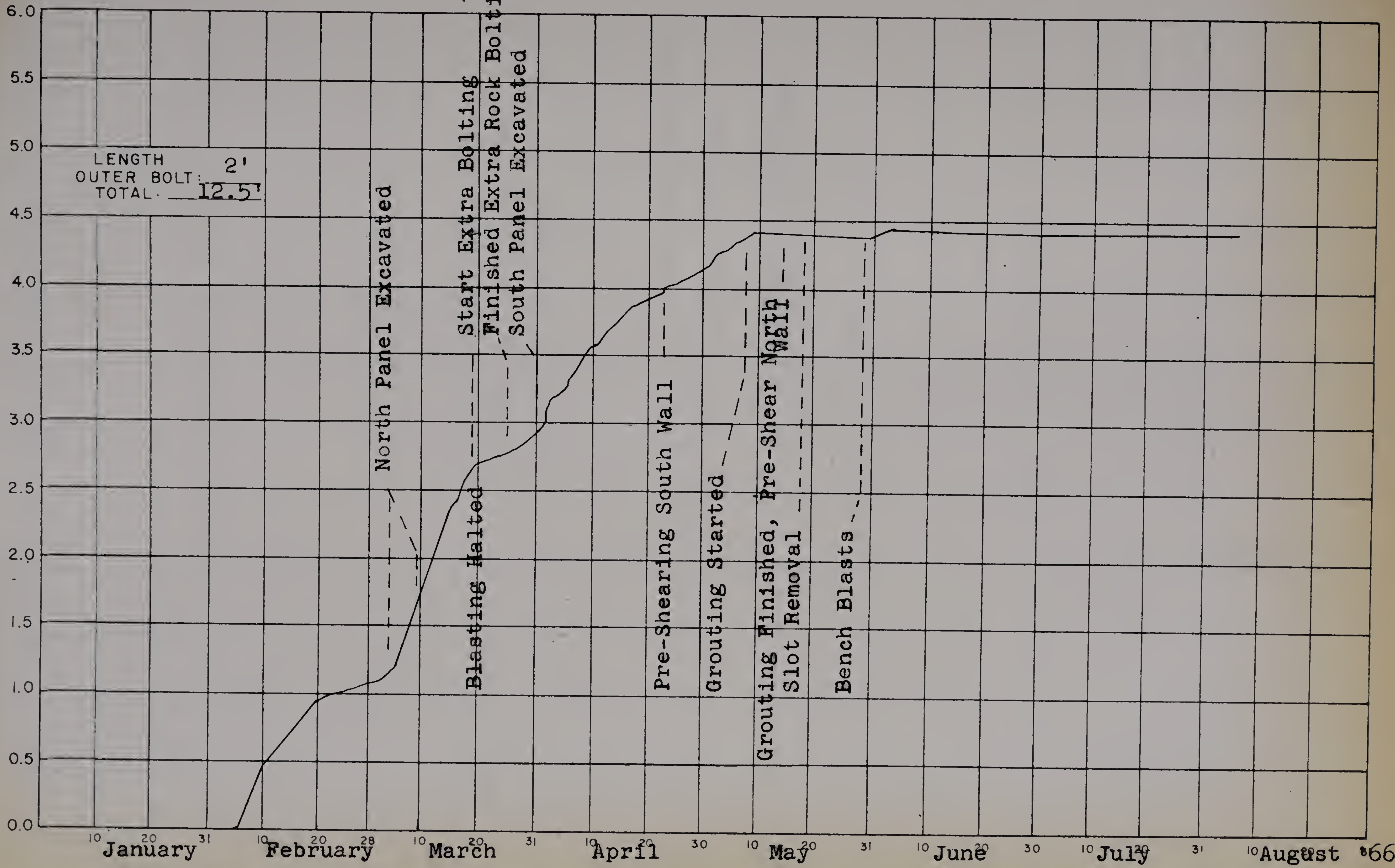


PORTAGE MOUNTAIN DEVELOPMENT

ROCK RELAXATION GAUGE

POWERHOUSE ROOF ARCH @ 709-0

FIG. 21

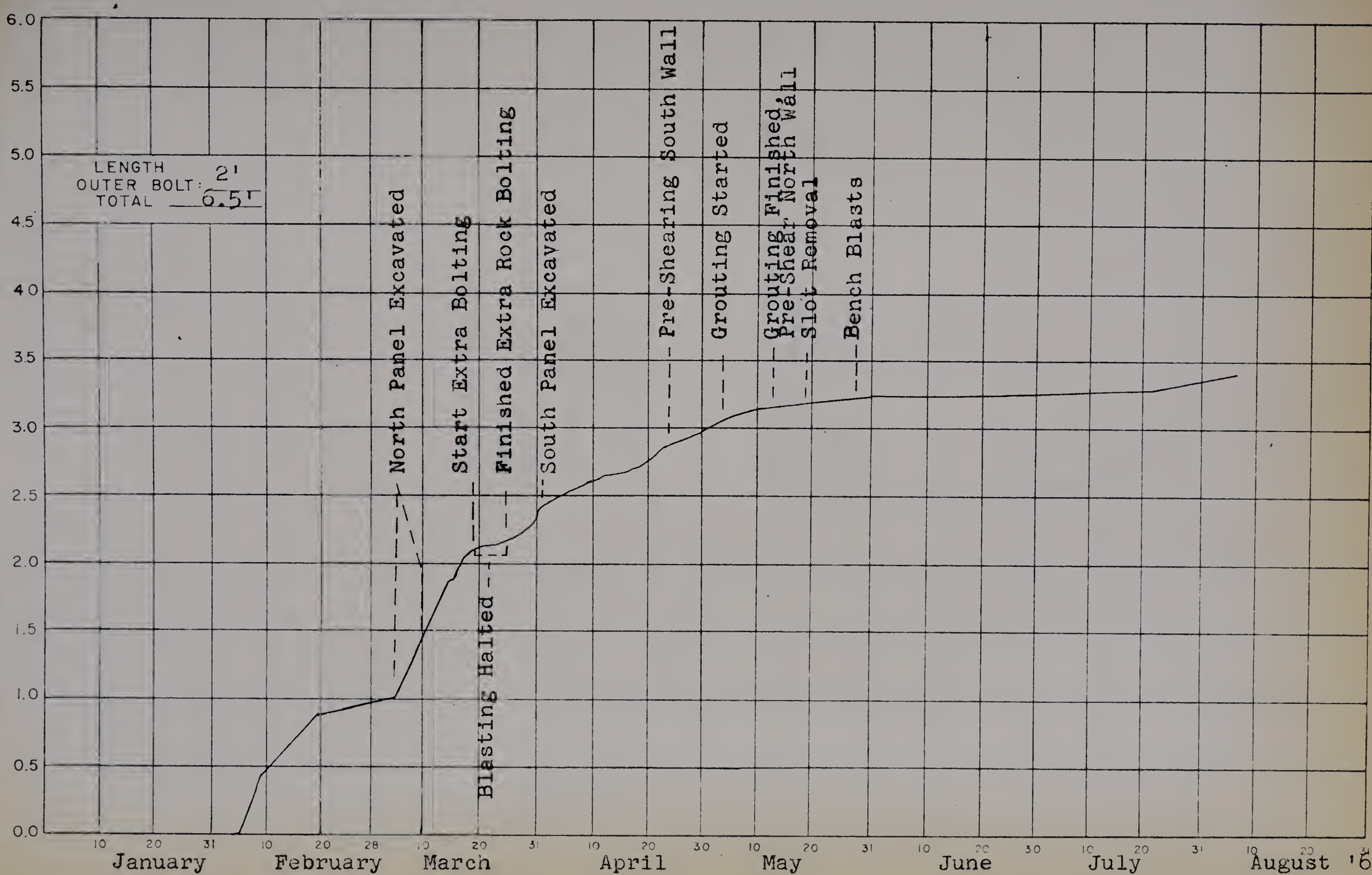


PORTAGE MOUNTAIN DEVELOPMENT

ROCK RELAXATION GAUGE

POWERHOUSE ROOF ARCH @ 710-0

FIG. 22

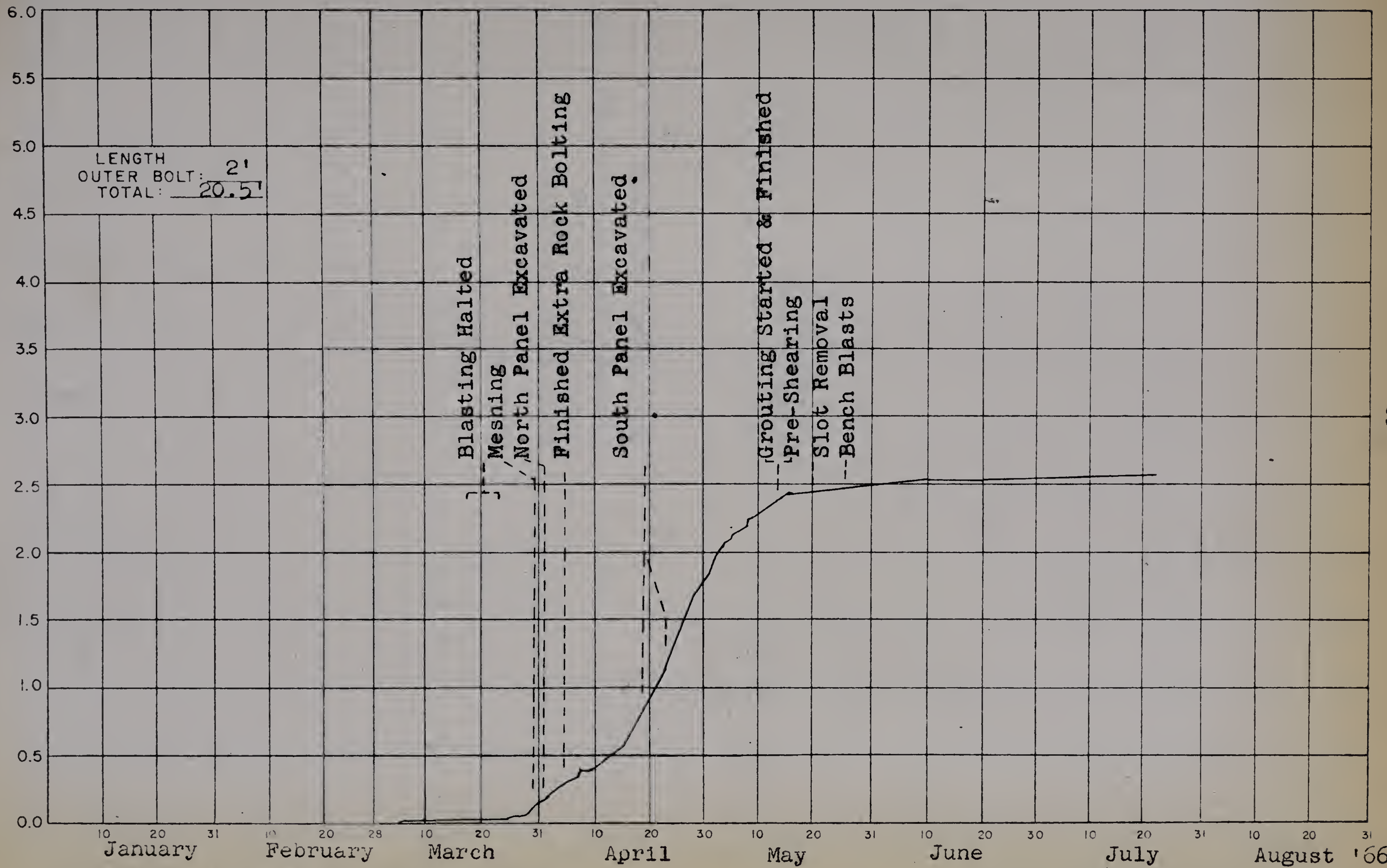


PORTAGE MOUNTAIN DEVELOPMENT

ROCK RELAXATION GAUGE

POWERHOUSE ROOF ARCH @ 712-1S

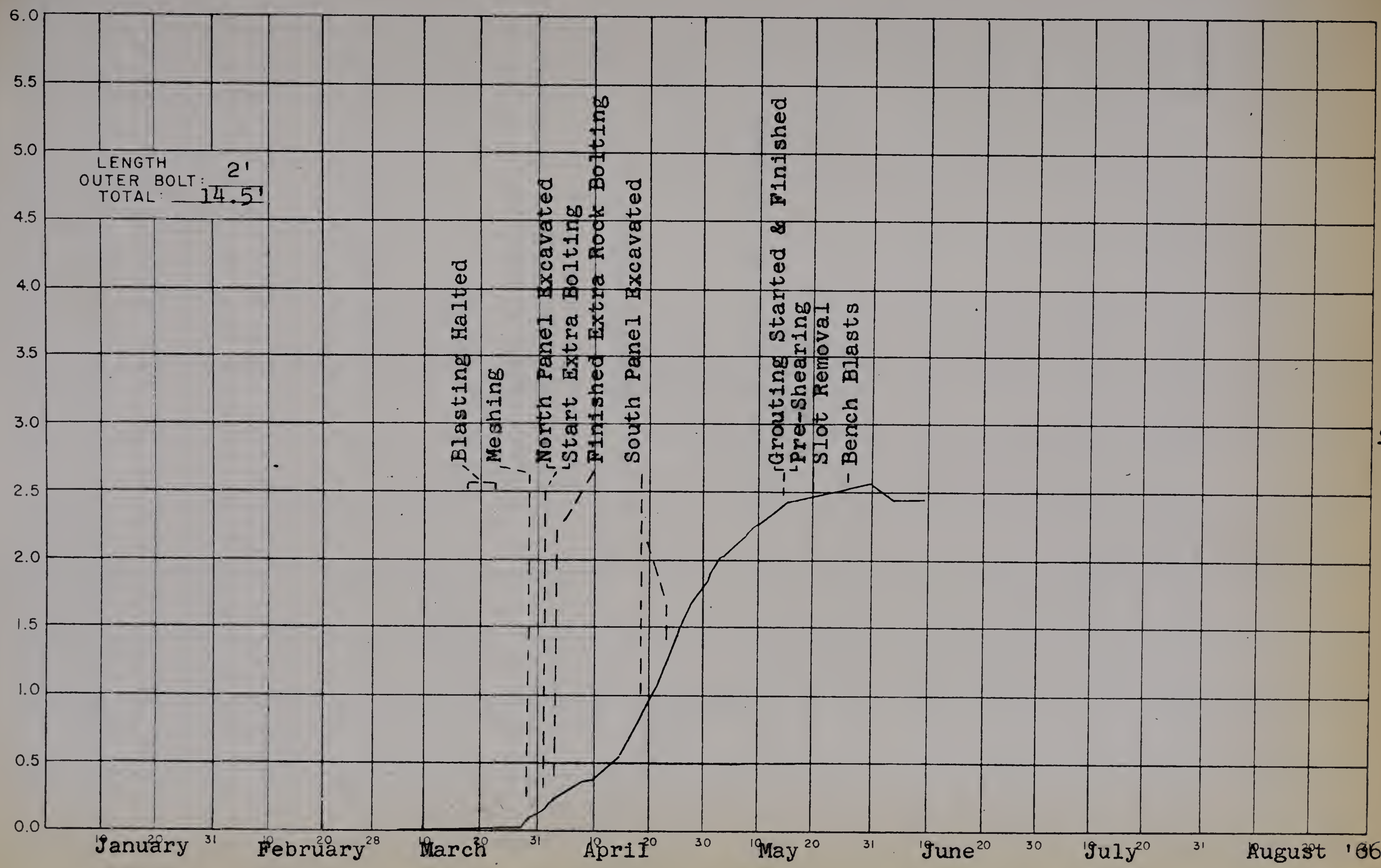
FIG. 23



PORTAGE MOUNTAIN DEVELOPMENT ROCK RELAXATION GAUGE

POWERHOUSE ROOF ARCH @ 804-1N

FIG. 24

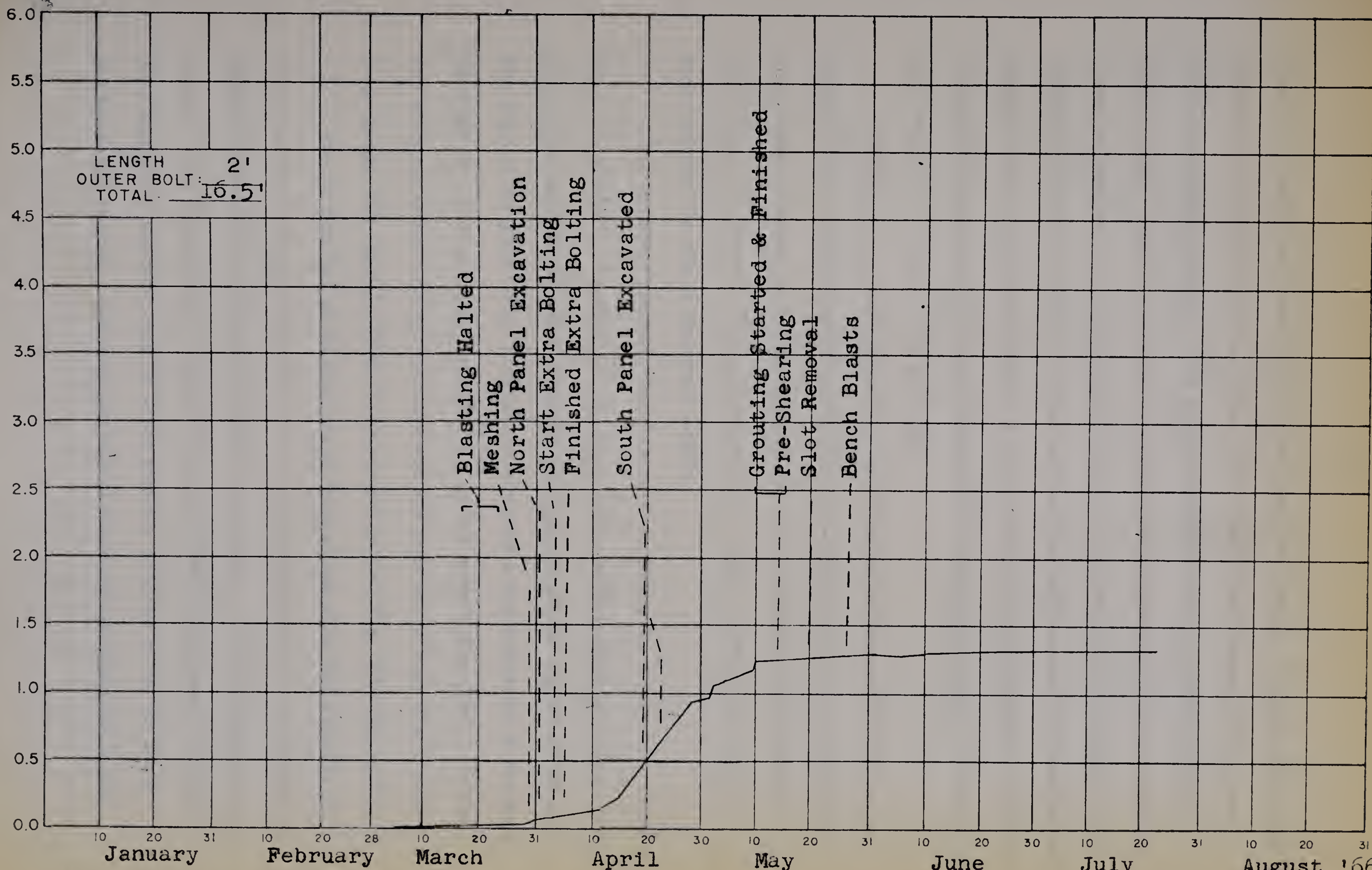


PORTAGE MOUNTAIN DEVELOPMENT

ROCK RELAXATION GAUGE

POWERHOUSE ROOF ARCH @ 805-2N

FIG. 25



PORTAGE MOUNTAIN DEVELOPMENT ROCK RELAXATION GAUGE

POWERHOUSE ROOF ARCH @ 806-1N

FIG. 26

directly related to arch blasting conducted near each gauge. A steady downward movement of the arch began soon after installation of each gauge and continued with two notable exceptions. The gauge movements virtually ceased during a blasting delay, March 18 to March 25, and ceased altogether with the completion of arch excavation in the second week of May. This was true of all gauges in the southeast half of the chamber. To the northwest the arch movements diminished considerably after the full arch opposite a gauge had been opened for about two weeks. During this time excavation had proceeded 60 to 70 feet further away. Minor movement occurred, however, until early May.

Although arch blasting definitely affected the movements recorded by the gauges, the pre-shearing and excavation of the underlying bench, as carried out at the times indicated on the graphs, had no marked tendency to accelerate or retard these movements. The supplementary rock support installed in certain areas retarded the rate of movement slightly.

One important comment should be made regarding the shape of the displacement - time graphs. Readings of the gauges were generally made on a daily basis rather than immediately before and after blasting in a certain portion of the arch. The slope of the graphs (Fig. 27a) are therefore not a true indication of the movement pattern which probably resembled a step-like pattern (Fig. 27b). In the few instances where readings were obtained just prior to and then after a blast (e.g. gauge 709, April 1, 1966) this movement pattern (Fig. 27b) was confirmed.

Results from four of the five sets of three gauges show that virtually all the movement measured occurred within 14.5 feet of the arch and that this strain was distributed fairly uniformly throughout the distance. The fifth gauge set at station 7 + 10 showed that 1.4 inches of movement occurred between 12.5 and 20.5 feet above the arch, most of which probably occurred at the N5 Shale - N4 Sandstone contact. Just over one inch of movement was measured in the

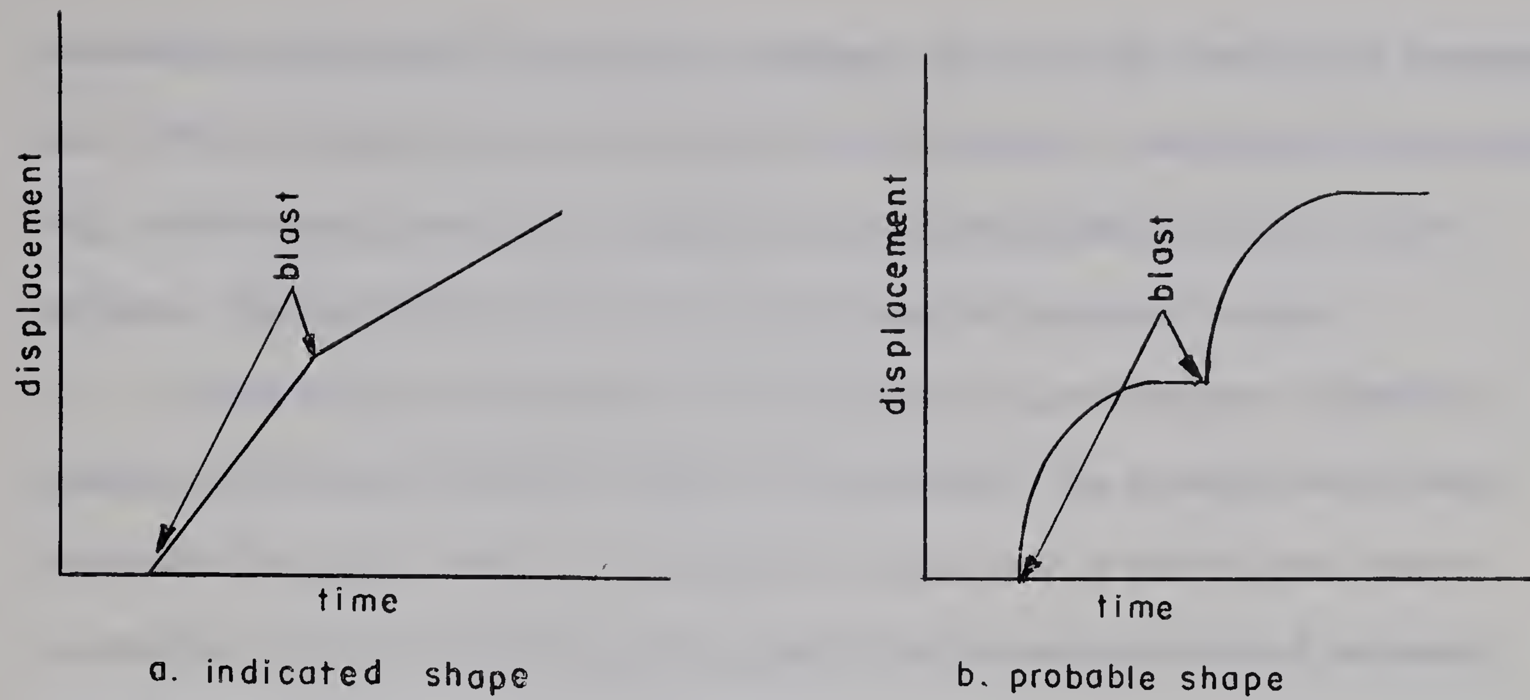


FIG. 27 SHAPE of TIME-DISPLACEMENT GRAPHS

interval between 6.5 and 12.5 feet, with by far the most movement (3.3 inches) in the lower 6.5 feet.

The structural rock bolts were grouted during April and May 1966. To ensure that inadvertent grouting of a gauge was not responsible for the lack of recorded movement at this time, three additional relaxation gauges were installed after completion of the grouting (Fig. 19). These last three gauges were read until August 1966 and recorded only minor creep of the arch.

Seismophone

A portable seismophone unit, consisting of a geophone, amplifier, earphones and tape recorder, was used during the last half of the arch excavation. This instrument, developed by an American insurance firm in conjunction with the United States Bureau of Mines (Crandell, 1955), detects microseismic activity in rock undergoing relaxation or readjustment.

Horizontal holes five feet long were drilled at distances of 100 feet in the northern haunch or side headings of the powerhouse for use as measuring stations. These stations were all at the invert level except for one located near the top of the arch at station 7 + 00. Seismophone readings could only be taken during

the rather rare periods of construction stoppage, thus severely limiting the instrument's use. The rock noise measured consisted of individual pops, sometimes hollow sounding, which were counted by an observer during listening periods of five to ten minutes. Tape recordings of the noise were made for permanent record.

Little activity was observed, even at measuring stations near relaxation gauges which showed relatively high rock movements. The average rate of noise was under five pops a minute, exceeding this value only at stations near recent excavation. The low activity again suggests that the major amount of movement probably occurred at the instant of blasting.

Drill Hole Probing

Eight vertical diamond drill holes were cored in the powerhouse arch during March 1966 in order to observe the physical state of the rock immediately overlying the partially completed arch. Three holes were drilled from staging at the top of the arch at stations 5 + 62, 6 + 60 and 7 + 30. Five holes were collared near the north haunch and drilled from the invert at stations 2 + 50, 4 + 00, 6 + 85, 7 + 00 and 7 + 15. In November 1966 three additional vertical holes were cored through the concrete arch along the centreline of the powerhouse at stations 3 + 00, 5 + 00, and 7 + 20.

Figure 28 (in pocket) is a stratigraphic section prepared from the drill hole logs. The elevation of the N4 Sandstone - N5 Shale contact was found to be fairly constant with two exceptions. At station 2 + 50 a facies change raises the contact almost three feet; at station 7 + 00 there is a sudden downward flexure followed by an upward rise in the strata which continues south-easterly for the length of the powerhouse. The final significant factor to be noted from this geologic section is the northwesterly increase in the amount of siltstone and sandstone in the N5 Shale above the arch.

The three vertical core holes (NX) drilled along the centreline of the

chamber in March 1966 provided a limited visual observation of the rock immediately above the arch. Horizontal fractures could be seen in the rock immediately above the collar of each hole and an attempt was made to measure these observed cracks and those suspected above them. Measurements were taken using a graduated 16 or 20 foot wooden loading stick with a nail probe attached to one end. Readings were taken at two different times in April 1966 and the results are given in Tables 10, 11 and 12.

The results of this probing may not be used to estimate directly the amount of rock relaxation which occurred between April 6 and April 19, 1966, for slightly different methods of measurement were used. In the first set of readings extreme care was taken to try to distinguish between fractures in the rock and chipping of the side of the hole during drilling (Fig. 29). This precaution was not taken for the April 19 readings.

Table 13 gives an interesting comparison of the total amount of fracturing measured by the probe and the movements measured by adjacent gauges. The data suggests that 60 to 80 per cent of the gauge movements recorded to April 6 can be accounted for by the opening of fractures. The results of April 19 readings show that the amount of fracturing exceeds the gauge movements probably due to mistaking chips for fractures.

As seen in Tables 10, 11 and 12 the limit of fracturing appears to be 12 to 14 feet above the collar of the hole. This agrees with the relaxation gauge information of the sets of gauges. It was also noted in the probing that two slight offsets (0.5 - 0.75 inches) had occurred in the strata near the collars of two of the holes after drilling. The underlying rock moved towards the southern side of the chamber relative to the overlying rock.

TABLE 10

Drill Hole Probing, Station 5 + 62

Distance From Collar	Estimated Size of Crack April 6/66 16' Probe	Estimated Size of Crack April 19/66 20' Probe
1' 6"	fairly tight	NR*
1' 8 1/2"	NR	1/8"
1' 9"	fairly tight	1/8"
7' 0"	fairly tight	1/8"
8' 10 1/4"	NR	fairly tight
9' 0"	fairly tight	NR
9' 2"	1/8"	1/8"
9' 9 1/2"	NR	1/8"
10' 0"	failry tight	1/8"
10' 9"	fairly tight	NR
11' 0"	fairly tight	3/8"
11' 10 3/4"	NR	1/4"
12' 0"	fairly tight	fairly tight
16' to 20'	no measurements	no fractures
Estimated Total**	5/8"	1 1/2"

*NR - Not recorded

**Fairly tight crack taken to be 1/16" wide

TABLE II

Drill Hole Probing, Station 6 + 60

Distance From Collar	Estimated Size of Crack April 6/66 16' Probe	Estimated Size of Crack April 19/66 20' Probe
1' 4"	1/8"	NR*
1' 8"	1/8"	1/4"
2' 0"	1/4"	NR
2' 1 1/2"	1/4"	3/8"
2' 2 1/2"	1/8"	NR
2' 6"	1/8"	3/8"
6' 6"	NR	fairly tight
6' 9"	1/16"	1/4"
6' 11"	1/8"	NR
9' 4 1/4"	NR	1/8"
10' 0"	tight	1/4"
10' 2"	tight	NR
10' 5"	tight	NR
11' 4 3/4"	NR	fairly tight
12' 6 1/2"	NR	1/8"
13' 5"	tight	1/8"
15' 0"	NR	1/4"
16' to 20'	-	no fractures
Estimated Total**	1 7/16"	2 1/4"

*NR - Not recorded

**Tight, fairly tight cracks taken to be 1/16" wide

TABLE 12

Drill Hole Probing, Station 7 + 30

Distance From Collar	Estimated Size of Crack: April 6/66 16' Probe	Estimated Size of Crack: April 19/66 20' Probe
0' 8"	1/8"	3/8"
1' 1"	fairly tight	1/4"
1' 10 1/2"	fairly tight	fairly tight
2' 10"	fairly tight	1/8"
3' 4"	1/8"	3/8"
3' 5 1/2"	1/8"	NR*
4' 3"	3/16"	7/8"
4' 7 1/4"	NR	3/8"
4' 8 1/2"	tight	NR
4' 11"	tight	NR
7' 2"	tight	NR
7' 3 1/2"	NR	1/4"
7' 5"	tight	NR
7' 7"	tight	NR
8' 7 3/4"	NR	1/8"
8' 9 1/2"	tight	NR
9' 7 1/4"	NR	fairly tight
10' 6"	NR	fairly tight
11' 6"	NR	1/8"
11' 8"	tight	1/8"
12' 0"	NR	7/8"
12' 3"	tight	3/8"
12' 9"	NR	fairly tight
12' 10 3/4"	NR	fairly tight
13' 0"	NR	tight
14' 4"	tight	NR
Estimated Total**	1 3/16"	4 5/8"

*NR - Not recorded

**Tight, fairly tight cracks taken to be 1/16" wide

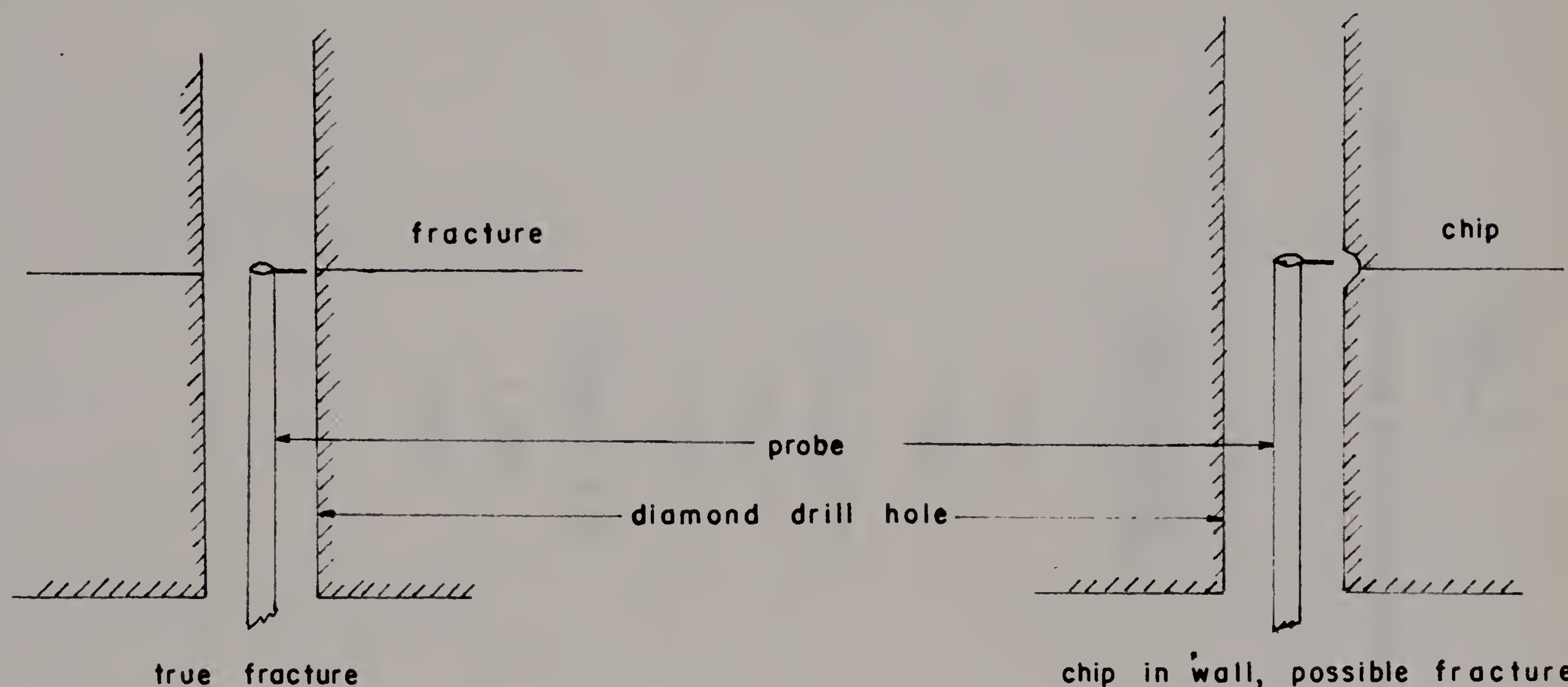


FIG. 29 DRILL HOLE PROBING

TABLE 13

Comparison of Gauge & Probe Measurements

	Gauge 5 + 81 Inches of Movement	Probe 5 + 62 Inches of Opening
April 6	0.917	0.6
April 19	1.050	1.5
	Gauge 6 + 69	Probe 6 + 60
April 6	1.725	1.4
April 19	2.100	2.3
	Gauge 7 + 32	Probe 7 + 30
April 6	2.000	1.2
April 19	3.083	4.6

Absolute Movement Measurements

As indicated above, the rock movements measured by the relaxation gauges are relative movements only; although if the relaxation in the massive N4 Sandstone in which the gauges are anchored is considered negligible, the recorded values should approximate the total downward movement. An attempt was made to check this assumption during the latter stages of excavation by precise survey methods. As with the seismophone, periods of measurement were very limited. There was no evidence, however, to suggest downward rock movements in excess of those measured by the relaxation gauges. In addition, horizontal measurements taken during April - May 1966 indicated no inward wall movement resulted from excavation of the bench slot (Fig. 18, stage 5) during this time.

Additional Support

On the basis of the available observations and measurements outlined above, supplementary rock bolts, mesh and strapping were installed in various parts of the arch in addition to the basic support specified. The basic pattern as specified by I.P.E.C. called for 58,000 linear feet of grouted one inch diameter rock bolts, 14 to 20 feet long, installed at a five foot spacing. The contractor, however, chose to change the basic pattern to a four foot spacing except for approximately 100 feet at either end of the powerhouse (Fig. 30).

From station 3 + 40 to station 4 + 10, the only section of the arch in the northwest part of the chamber to show appreciable movement (one inch), approximately 120 additional bolts, 20 feet long, were installed. More extensive bolting was carried out in the vicinity of the powerhouse ramp from station 4 +40 to station 5 + 20. Here the basic pattern bolts were not able to bear the extra load resulting from the lack of arch support on the south side of the chamber. A three foot pattern was used to support this region. By far the greatest number of

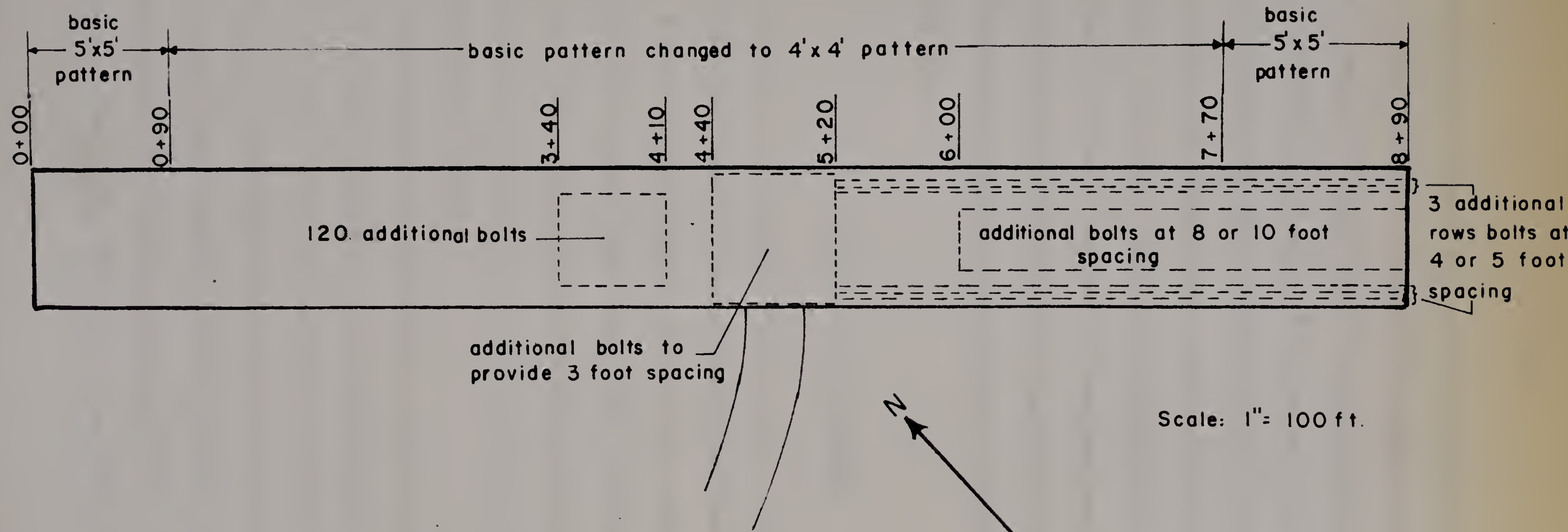


FIG. 30 Plan of Powerhouse Arch

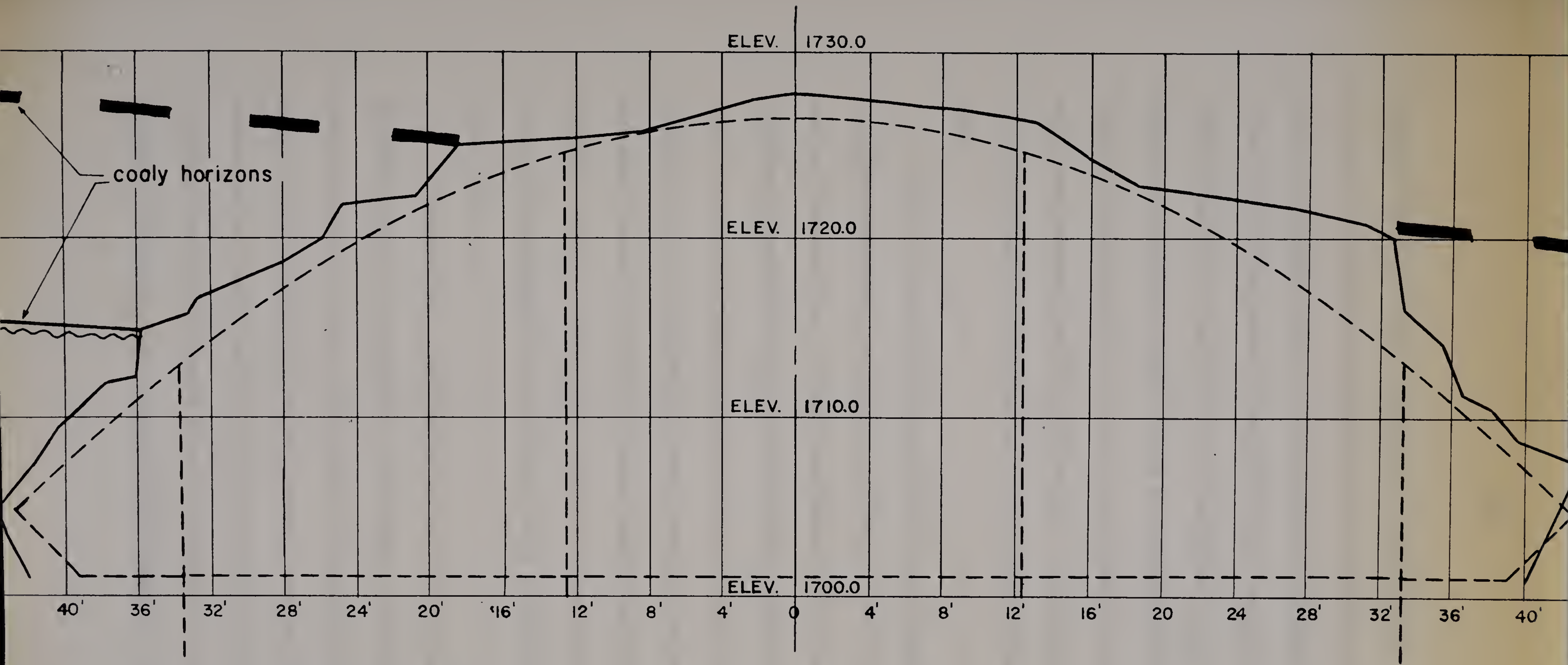
SUPPLEMENTARY ROCK BOLTS

additional bolts were added to the arch in the southeast end of the powerhouse (station 6 + 00 to station 8 + 90) where the maximum movements were being recorded. The central third of the arch received additional rock bolts every eight to ten feet for the full length of this section. Also in this interval three rows of supplemental bolts were installed in each of the rock "shoulders" occurring on the north and south sides of the arch (Fig. 31), which were caused by overbreak to coal seams. Supplementary shoulder bolts were eventually installed back to station 5 + 20. All of these areas were extensively meshed and strapped during the installation of the supplemental rock bolts. A total of 103,000 linear feet of grouted one-inch-diameter rock bolts was installed in the powerhouse arch.

Nature of Arch Movement

Because of the anisotropic state of the rock, it is difficult to form definite conclusions regarding the nature of the arch relaxation recorded by the gauges. A comparison of the measurement readings in the east and west ends of the powerhouse, however, gives some insight into this relaxation. Rocks above the arch are thought to have moved by the following mechanisms: (1) homogeneous elastic and elastico-viscous expansion, (2) buckling and wedging, and (3) opening of horizontal tension cracks.

Elastic and elastico-viscous dilation probably contributed the least amount to the total movement measured by the relaxation gauges. These gauges were installed at the face of the centre heading up to eight hours after blasting and most of the truly elastic deformation would have occurred by then. Elastico-viscous movements (time-dependent strain) occurred for some time after this but these movements are believed to have been small. It is interesting to note that a structural flexure (Fig . 28) occurs in the vicinity of the highest arch



GEOLOGY REFERENCE: figure 8

POWERHOUSE ROOF ARCH

TYPICAL POST-EXCAVATION CROSS-SECTION

SCALE: 1/8" = 1'-0"

NOTE: SECTION LOOKING SOUTHEAST

FIG. 31

movement (station 7 + 00). The stress conditions at this point would probably be higher than the surrounding rock and resulting dilation after excavation would probably be slightly higher. At the Poatina underground powerhouse, however, a downward roof movement of only 0.07 inches was accounted for by elastico-viscous expansion in rock of similar lithology and in situ stress conditions (Endersbee and Hofto, 1963). During the construction of the Sir Adam Beck Niagara Generating Station in flat-lying sedimentary rock, vertical arch movements were extremely small (0.050 inches). No in situ stresses were measured in this case although it is possible that high residual stresses exist in the area (Coates, 1963).

That the horizontal stresses at the Portage Mountain damsite have resulted in bedding-plane slip is evidenced by offsets (up to 0.75 inches) observed in two core holes and one rock bolt hole above the arch during March - April 1966. This shearing movement together with buckling of the strata under the influence of the horizontal compressive stresses also contributed to the arch movements but this mechanism is not believed to have contributed the major part. While the horizontal stresses are higher in the east end of the chamber than the west end (Fig. 11), the large differences in maximum movement between the two areas (5.8 inches as opposed to one inch) cannot be entirely due to the difference in stress levels.

The main factor contributing to the arch movements, and more specifically the differential arch movements, would appear to have been dynamic vertical loading caused by blasting. One theory of blasting (Duvall and Atchison, 1957) is that most of the fracturing in rock exposed to a blast is the result of compression waves being reflected into tension waves at a free surface; the rocks fail in tension, the tensile strength of rocks being much less than their compressive strength. At Portage Mountain the stress distribution in the rocks above the

arch during blasting was probably very complex. Compression waves, reflected from the overlying N5 Shale - N4 Sandstone contact as tension waves, were probably a factor in the formation of the fracturing observed above the arch.

The east end of the powerhouse was excavated in an irregular manner and the arch was subjected to dynamic loading over a long period of time. At any one point in the excavation sequence (Fig. 19), the central heading advanced far ahead of the other faces. Following the excavation of the side faces on the north side of the powerhouse, the excavation of the south faces was carried out up to two months behind the central heading. Each episode of side blasting subjected the arch to severe vertical loading and kept the rock active such that it relaxed considerably. In the west end of the powerhouse the excavation was more rapid and uniform, and the dynamic loading had less effect.

Another factor contributing to the difference in total movement between these two areas is the change in lithology from a predominantly shale section in the southeast end to a sandy shale in the northwest end (Fig. 28). The shale sequence contains more thin coal lenses giving this rock a lower tensile strength. The formation of horizontal tension fractures would thus occur more readily in the east end of the powerhouse. The difference in lithology is important as over one inch of movement occurred at station 7 + 10 before the excavation of any side or haunch faces (Fig. 21). This suggests that the movements recorded in the southeast end in the powerhouse would have been higher than in the northwest end even with a satisfactory excavation sequence.

As indicated above, the opening of these tension fractures appears to have been responsible for the majority of the arch movements. There is recent evidence, however, to suggest that the openings recorded by drill hole probing (Tables 10, 11 and 12) may not be quite as extensive as first thought. The contractor cored three vertical holes along the centreline of the arch in November 1966. The

puzzling factor is that the core from the hole drilled at station 7 + 20 contained only minor grout in an area where almost six inches of relaxation was recorded.

The total amount of tension fractures could be found definitely by using a borehole camera or periscope to observe the walls of a borehole rather than having to estimate the fractures by touch. If the extent of fracturing is considerably less than that indicated above, then more movement must have occurred due to a wedging or buckling action. In this case the horizontal stresses must have been an important factor in the arch movements.

Plate 3 Arch at Station 4 + 20 Showing Overbreak to Coal Seam which occurred for Full Length of Powerhouse

DESIGN ANALYSIS

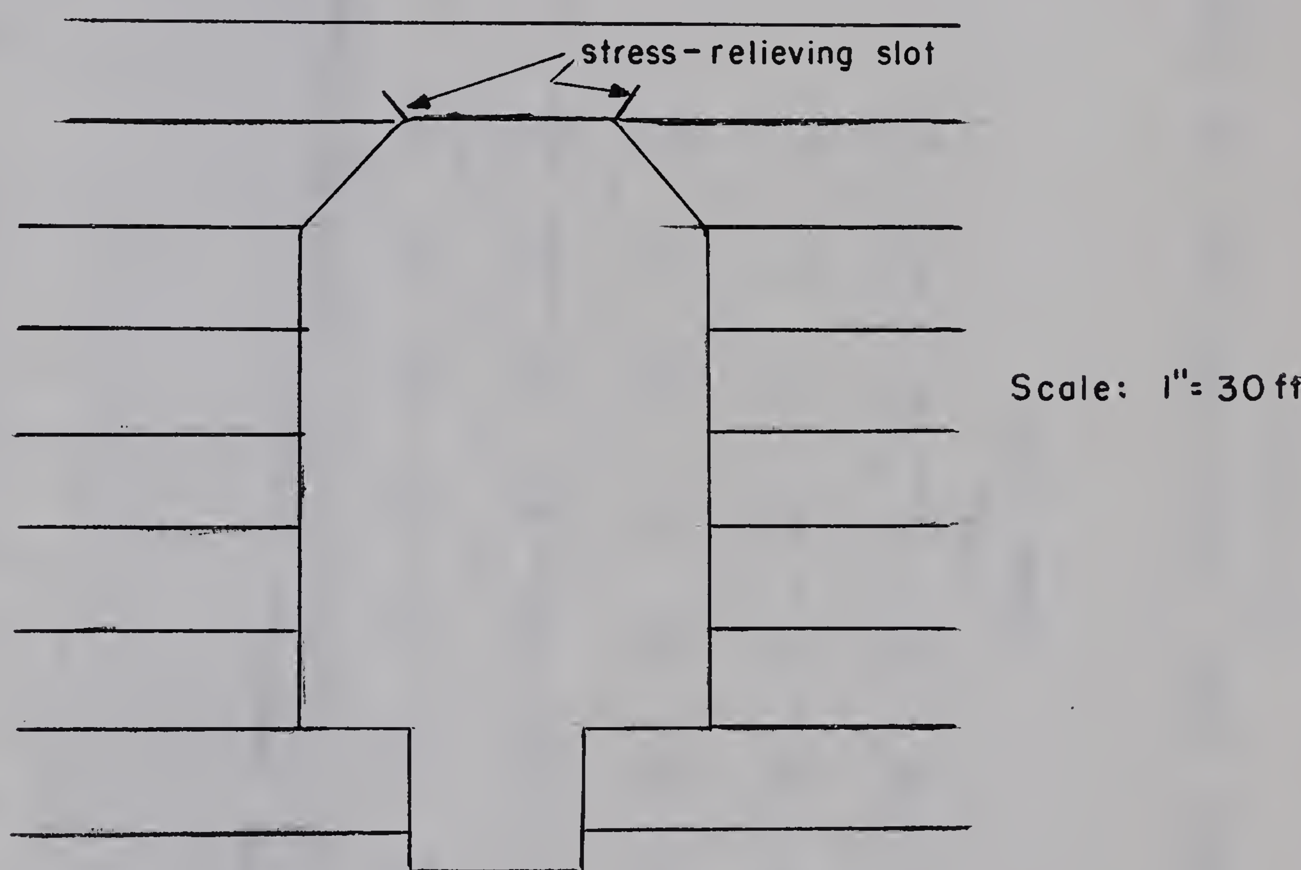
With unprecedented structures of the size of the Portage Mountain powerhouse the design of the powerhouse roof and its support is difficult because the designers are unable to draw on past experience to assist them in certain aspects. Instead they must attempt to predict conditions which have never been encountered before. A basic design must be flexible, therefore, in order to deal with unforeseen or changed conditions during actual excavation. Flexible or not, no design can be expected to be perfect under these circumstances. In hindsight, modifications can be expected, and on this basis, some recommendations and opinions are presented below.

Shape of Powerhouse Roof

Theorectical stress concentrations around openings of various shapes have been outlined by Obert _et al_ (1960). For an opening in isotropic rock and a hydrostatic stress field, a roof with a semi-elliptical or semi-ovaloid shape will result in the lowest concentrations of stresses. This is for an opening in which a width to height ratio either smaller or greater than unity is desired.

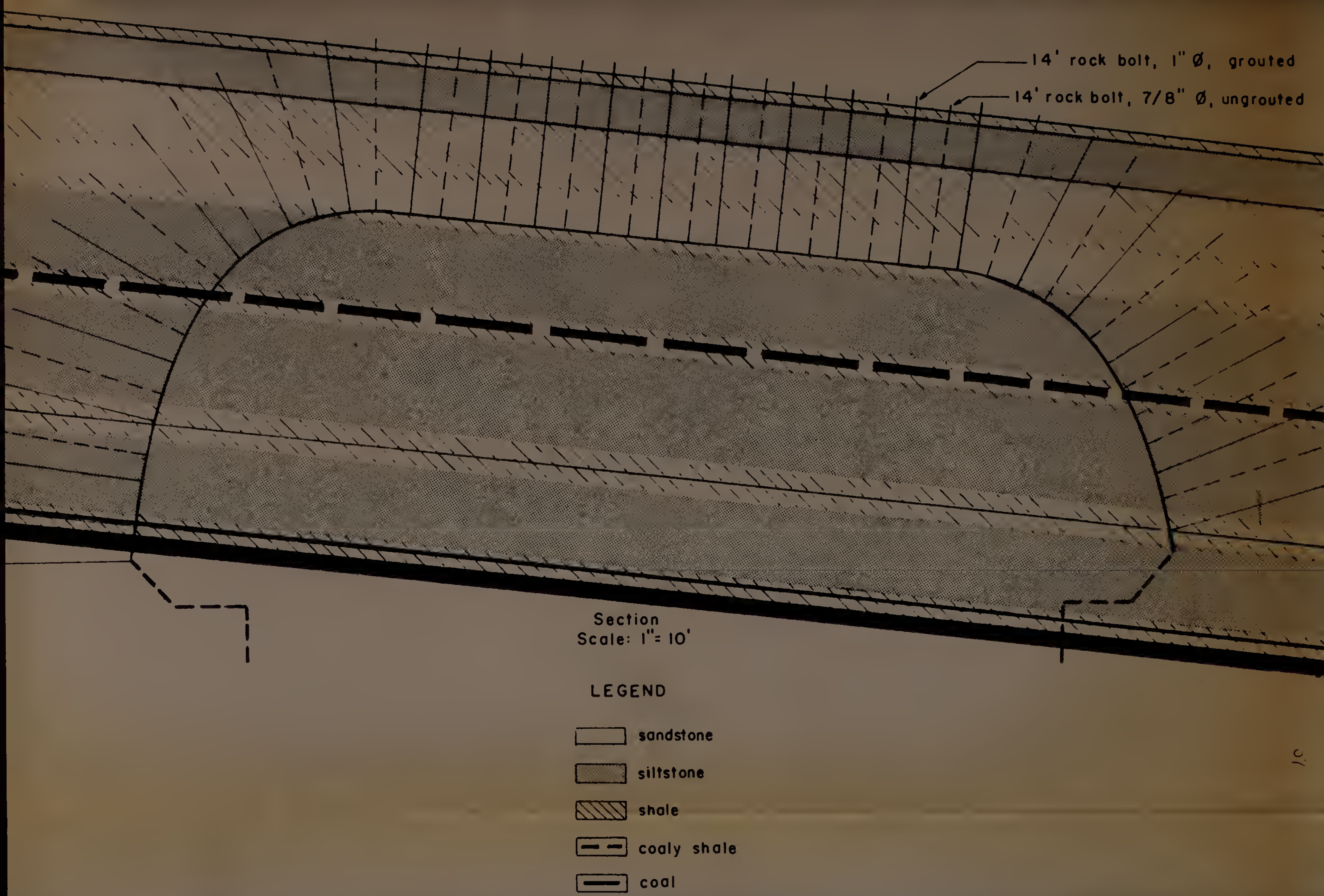
In practice, however, a semi-elliptical opening may be difficult to excavate. At Portage Mountain major overbreak to coaly horizons occurred because of the extremely low cohesion between the coal and underlying rock. Large blocks of shale or siltstone fell out under their own weight during excavation. Although "smooth blasting" technique (Langefors, 1959), was not used at the arch perimeter, it is doubtful that the overbreak (Fig. 31) could have been prevented if this technique had been specified. Endersbee and Hofto (1963, p. 196) state that even with cohesively bedded rock as at Poatina, a semi-elliptical roof is an unstable shape under conditions of high residual stresses.

At Poatina a trapezoidal roof shape was used with a flat back and steep sides (Fig. 32). Photo-elastic model studies of this shape revealed extremely high stress concentrations at the intersection of the haunch with the roof. Stress-relieving slots were drilled close behind the face to reduce the tangential stresses at the sharp corners of the excavation. In hindsight, at Poatina, Endersbee and Hofto (ibid, p. 209) state that an almost square underground opening would have been acceptable.



Final Design Cross-section
Poatina Underground Powerhouse(ref. Endersbee & Hofto)
Fig. 32

In hindsight at Portage Mountain a roof shape resembling a semi-ovaloid in cross-section is suggested (Fig. 33). The steep sides are necessary to reduce the overbreak to coaly horizons. The rounded corners are desirable as the stress-relieving required for a trapezoidal or square opening is a demanding and time-consuming process. Finally, experience at Portage Mountain has shown that near the centre of the opening, the roof tended to break parallel to bedding regardless of the blasting pattern used. The suggested shape is an example of making the opening fit the rock rather than attempting to make the rock fit the opening.



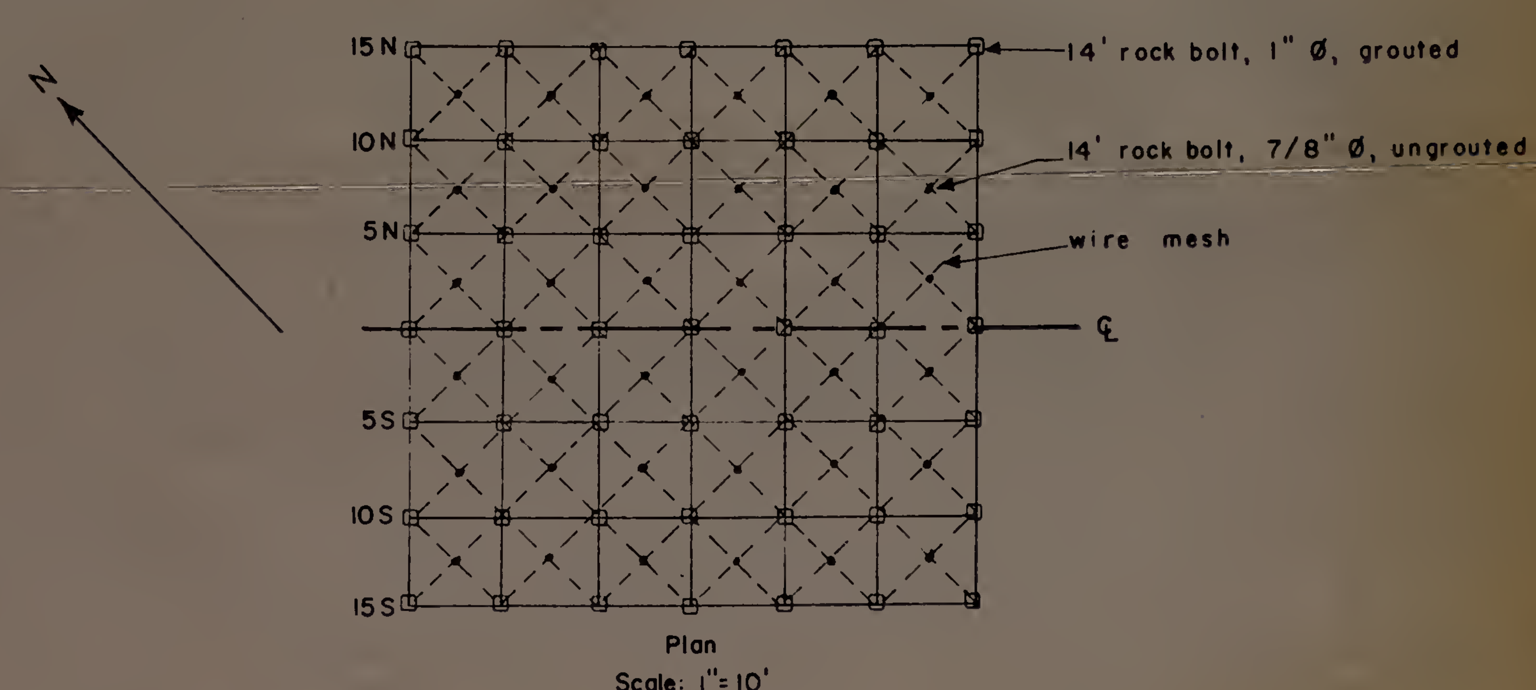


FIG. 33 PROPOSED ROOF CROSS-SECTION and ROCK BOLT PATTERN

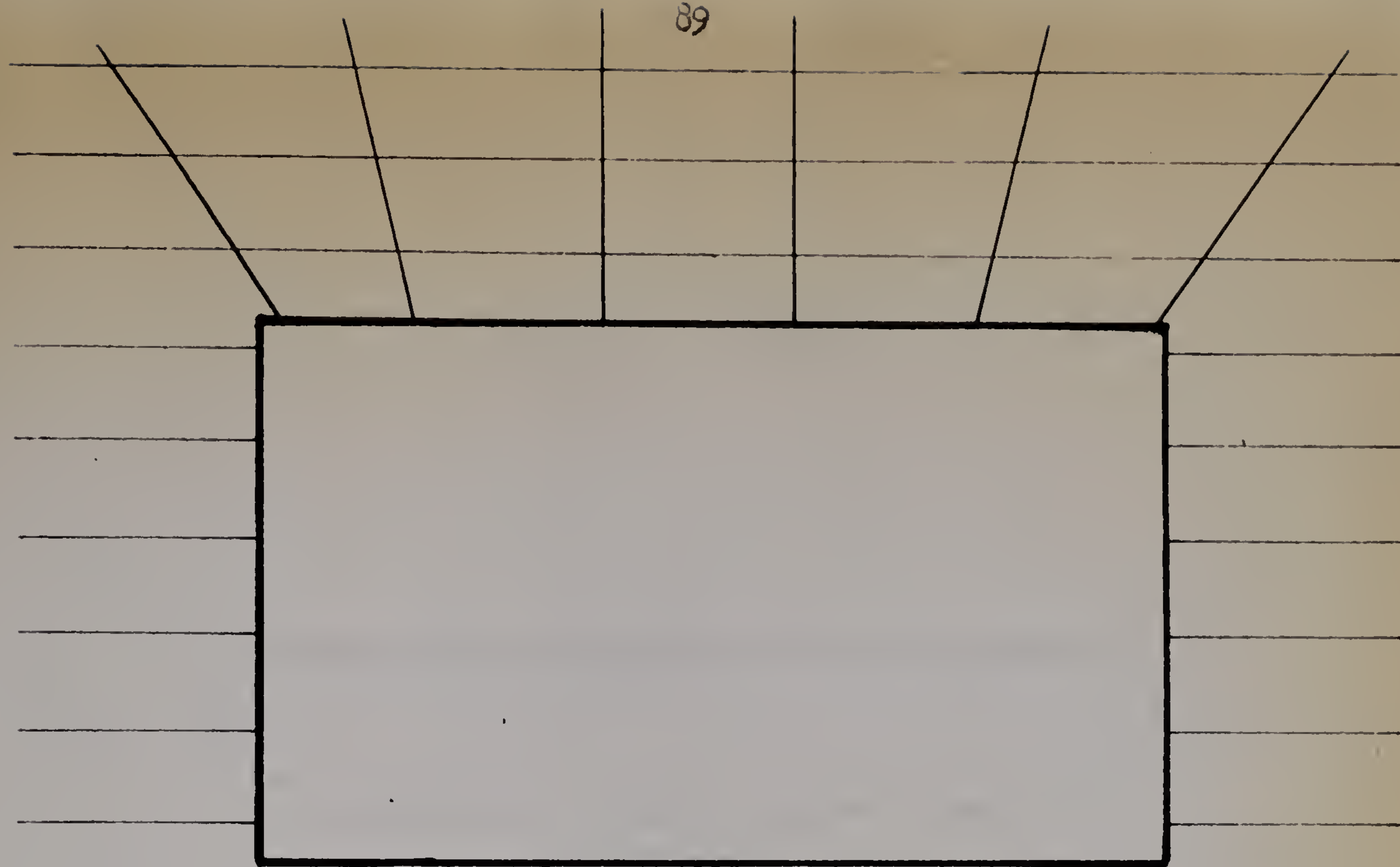
Support of Powerhouse Roof

Theory of Rock Bolting

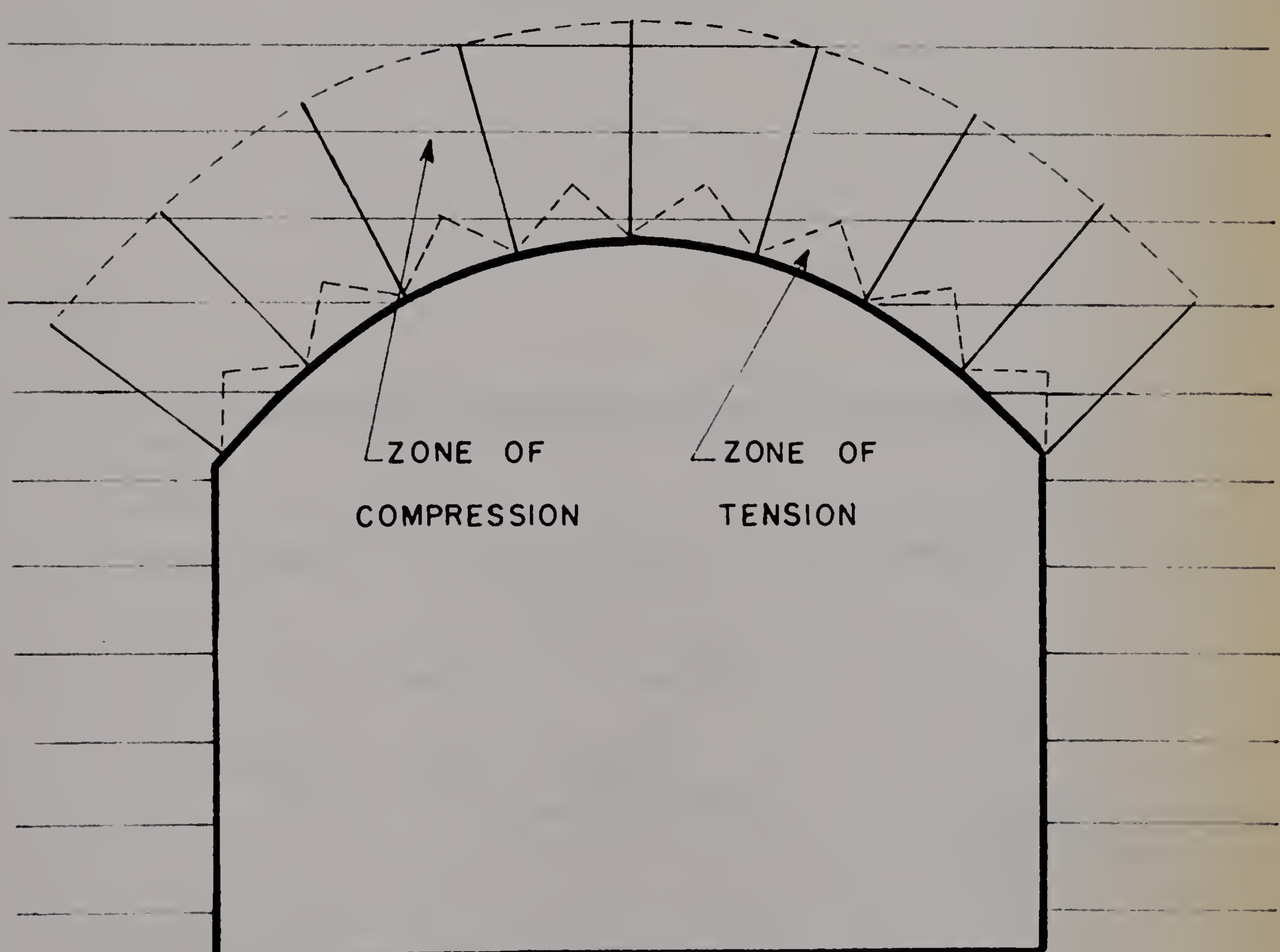
In the last ten years rock bolting has become widely used as a means of rock support in large underground civil engineering projects. The use of rock bolts at Poatina (Endersbee and Hofto, 1963), Tumut I and Tumut 2 (Pender et al, 1963) and at NORAD, Colorado (Underwood and Distefano, 1964) has been well documented. Rock bolts have also been successfully used in the underground powerhouses at Oroville Dam, California, and Boundary Dam, Washington, although no published data is known to the writer. Schmuck (1957) and Lang (1961) have discussed the theory of bolting but far more practical research is necessary before rock bolting changes from an art to a well understood science.

There are at least threee different mechanisms used to describe the action of rock bolts. The first is beam building in which bolts are installed in stratified rock to bind the individual layers together by increasing the friction between bedding planes causing the rock mass to behave as a single beam (Fig. 34a). The rock bolts, although exerting a compressive force on the rock, are themselves in tension. This beam must be able to support itself as well as resist the stresses that may be transmitted from the overlying rock.

A bolted mass of rock above an opening cannot be considered entirely analogous to a steel beam for the following reason. Whereas a steel beam has a high tensile strength below the neutral axis (Fig. 35a), rock mass generally does not due to the presence of joints, fractures or bedding planes. A better analogy may be to a uniformly loaded beam subjected to end loadings as in a column (Smith, 1963) (Fig. 35b). This analogy is appropriate to Portage Mountain where Hast and C-I-M Consultants showed the existence of large horizontal stresses in the rock.



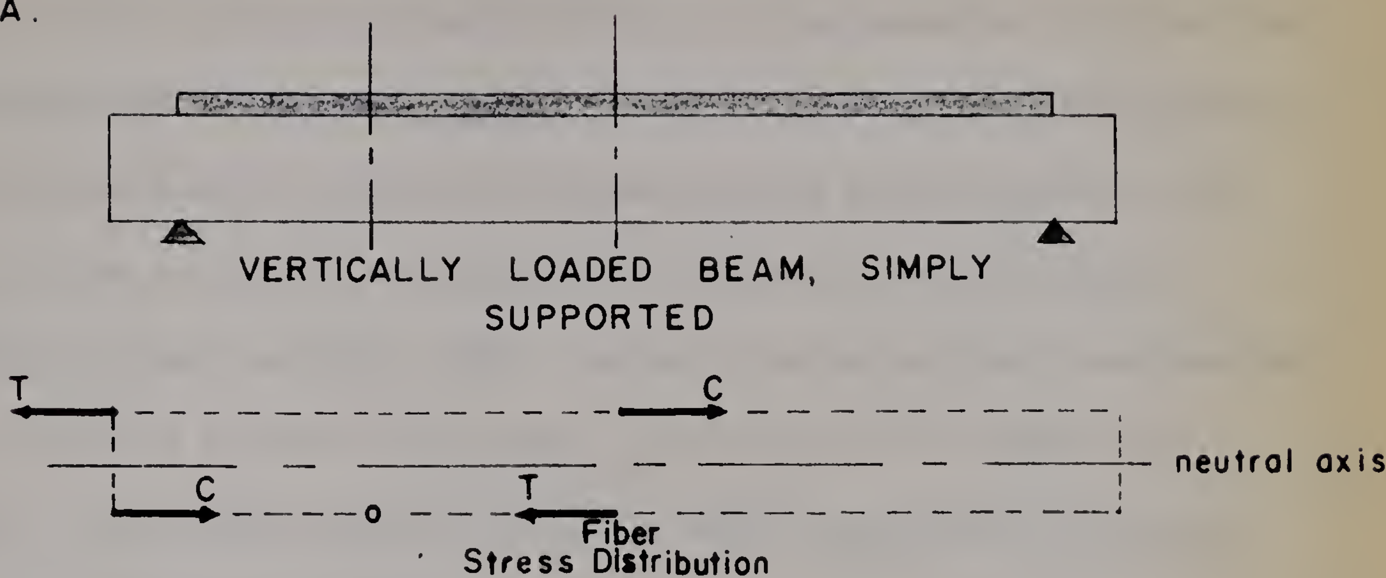
a. BEAM-BUILDING



b. ARCH REINFORCEMENT

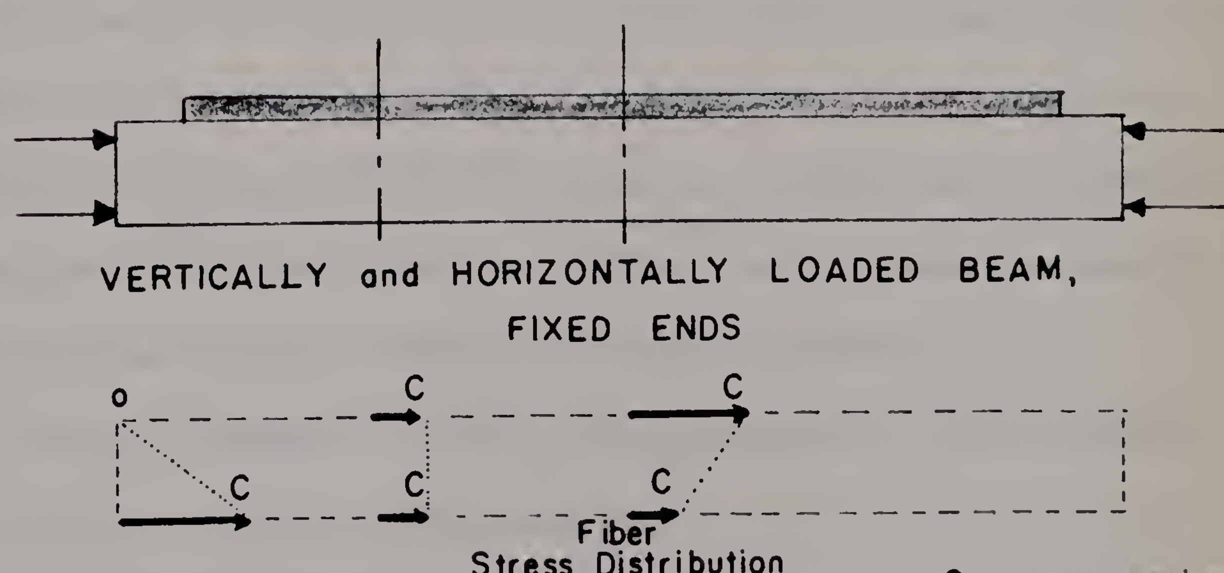
FIG. 34 MECHANISMS OF ROCK BOLTING

A.



VERTICALLY LOADED BEAM, SIMPLY SUPPORTED

B.



VERTICALLY and HORIZONTALLY LOADED BEAM, FIXED ENDS

C - compressive stress
T - tensile stress

FIG. 35 BEAM THEORY (after Smith, 1963)

The second mechanism is reinforcement and maintenance of an arched opening. During excavation, surrounding rock masses will decompress and tend to move into the new opening. If support in the form of rock bolts is installed soon after excavation this movement will be impeded. In addition, compressive stresses at right angles to the direction of the bolts are produced in the rock due to the longitudinal compression effect of the bolts. These compressive forces prevent pieces of loose rock from falling out of place and so retain the components in the structure. The stressed rock is thus able to act as a self-supporting structural unit (Fig. 34b). The length of the rock bolt, and thus the depth of the zone put under compression, is a function of the width of the opening. In the Snowy Mountains projects, results suggested the following relationship (Pender et al, 1963):

$$L = 6 + 0.004S^2 \qquad (5)$$

where L = length of bolt in feet and S = span of opening in feet.

Lang (1961) has outlined the photo-elastic studies at Tumut I, which established the basis for this mechanism of rock bolting. These studies have determined that the optimum spacing of rock bolts should not be more than one half of the length of rock bolts in order for a uniform zone of compression to develop at right angles to the bolt. With a spacing : length ratio of 1 : 3 the zone approximates two-thirds of the length of the bolt.

The third mechanism is that of simple suspension in which bolts are used to pin loose rock to overlying solid rock.

At Portage Mountain all three mechanisms were effective in supporting the roof of the powerhouse. The two important mechanisms appear to be beam building in the centre of the span and secondly, the reinforcement of the arched abutments (Fig. 31). Bolts were also used to suspend the rock shoulders resulting from the major overbreaks to coal horizons.

Design of Rock Bolt Patterns

The excavation of the powerhouse arch at Portage Mountain provides some useful evidence to consider in view of the design theories previously suggested for rock bolting programs. With regard to the length of the bolt selected for an opening, if equation (5) above were used the bolts for the powerhouse arch would have been 36 feet long. This length of bolt is both impractical to install and unnecessary. The depth of the decompression zone above the arch was only about 14 feet according to the relaxation gauge readings. On this basis a simple modification of equation (5) is proposed:

$$L = 6 + 0.004R^2 \qquad (6)$$

where R is 1/2 the span of the opening. This relation establishes bolt lengths of 14 feet in openings the size of the powerhouse. At the Portage Mountain powerhouse the increase in the length of the central bolts to 20 feet was a necessary modification due to the nature of the overlying strata.

The main question in the design of a rock bolt pattern is whether or not the rock bolts should be spaced so that each bolt supports the weight of the rock surrounding it. Coates (1965, p. 3-29) suggests an "ultimate design theory" in which each rock bolt should be able to support the tributary area of rock surrounding its entire length. He states that the design requirements could be reduced if experience showed that there was very small possibility of ultimate loads occurring.

Experience at Portage Mountain illustrates that the basic pattern of grouted bolts, one inch in diameter, 14 or 20 feet long, and spaced five feet apart would have been sufficient for the permanent support of the arch after excavation. The change to a four foot spacing (Fig. 30) was unnecessary. In the northwest end of the powerhouse relatively minor arch movements occurred, and in hindsight, the 120 supplementary grouted bolts which were installed

sometime later than the pattern bolts were also not required. In the southeast end of the powerhouse where the shalier lithology, slower excavation sequence and higher in situ stresses resulted in significant downward movement of the arch, it is more difficult to say definitely that a five foot pattern of rock bolts would have been adequate for the permanent support of the arch. However, it is important to note that the arch of the adjacent manifold no. 2 (Fig. 11) has been successfully supported using a basic five foot pattern of grouted rock bolts. Excavation of this manifold arch was completed in April 1967, almost a year after the powerhouse arch.

The five foot basic spacing, in agreement with Lang's suggestion that the spacing approximate one-third of the bolt length, is proper for arch building, but is such that each bolt in the basic pattern is not intended to support the surrounding rock. From Tables 14 and 15 an "ultimate design" would require one inch bolts at 3.5 foot spacing which would be an expensive and unnecessary bolting pattern.

Experience at Portage Mountain has also shown that in addition to the grouted rock bolts used for permanent support, wire mesh installed with ungrouted rock bolts is extremely necessary for the support of the roof during excavation. It is also important that this mesh be installed at the same time as the basic pattern bolting, which was not done initially for the powerhouse arch. The reason for the importance of mesh is shown in Figure 35b. Between the compression zones surrounding the rock bolts are triangular tension zones near the surface, in which rock will loosen under the efforts of dynamic loading. If unchecked this loose rock will gradually drop out and eventually undermine the structural rock bolts. This will render the bolts useless and conceivably could lead to continuing failure of the support system as one bolt failed, throwing additional weight on the adjacent bolts. Mesh, if installed immediately, will impede the breakup of the rock between the bolts and prevent it from dropping out.

TABLE 14

Basic Rock Bolt Patterns
Powerhouse Roof

Pattern	Quantity of Bolts Required	Maximum Load for Supporting 14 Feet Rock (lb.)[1]
5 Foot spacing	3380	54,250
4 Foot spacing	5100	34,720
3.5 Foot spacing	6600	26,580

[1]The density of shale at the Portage Mountain damsite is 155 lb./cu. ft.

TABLE 15

Mechanical Properties[1] of Rock Bolts

Diameter (inches)	Type	Minimum Yield Point[2] (lb.)	Minimum Breaking Load (lb.)	Installed Price ($ per foot)
1 (grouted)	Hollow	35,000	50,000	5.00
7/8 ungrouted	Solid[3]	30,000	44,000	3.50
3/4 ungrouted	Solid	21,500	32,000	2.75

[1] Manufacturer's data: ASTM A-306 Grade 80 Specifications

[2] Rock bolts are not intended to function above the yield point of steel

[3] Solid rock bolts are not easily grouted. If grouting of rock bolts is required on large scale, then one-inch-diameter bolts should be used.

It is suggested that the bolts used to support the mesh be essentially the same length as the basic pattern bolts. Moreover, they should be installed at such a spacing that the composite pattern is able to support the total weight of rock surrounding the bolts during the critical excavation stage. This support design is judged necessary in large excavations where dynamic loading is present over extended periods of time. Under these conditions there is a distinct possibility that large slabs of roofrock could transfer their entire weight to the rock bolts, at least locally, if not on an extensive scale.

From Tables 14 and 15, a 7/8th-inch-diameter bolts installed in the centre of the basic five foot pattern (equivalent to 3.5 foot spacing) would be necessary at Portage Mountain for the mesh installation with this design (Fig. 33). These mesh bolts would not be part of the permanent support, and thus not grouted, unless significant movements were recorded during the course of the excavation. Although the 7/8th-inch-diameter bolts required for this design would be slightly more expensive than the 3/4-inch-diameter bolts, eight feet long, which were used with the mesh at Portage Mountain (Table 15), the far more expensive process of going back to an area to install supplementary bolting would be eliminated.

Application of Rock Mechanics

The application of rock mechanics studies in the design of large undergrount openings such as the Portage Mountain powerhouse is recommended. Major stresses due to gravity loading or residual horizontal stresses cannot be resisted by rock bolts or similar support but instead must be resisted by the rock around the opening. The shape of the opening at any time during excavation must be designed so that regions of stress concentrations will not result in failure of either the surrounding rock or of the support systems. Therefore, the magnitude

and direction of the in situ stresses should be known and considered along with geological information. At Portage Mountain the pre-shearing and bench blasting immediately below the arch was a necessary step to prevent possible failure in the concrete arch caused by these in situ stresses.

During excavation of a large opening it is very important that the effects of forces on the rock immediately surrounding the opening are carefully observed in order to determine if the support design is functioning as intended. If these forces, which may be the result of dynamic loading or in situ stresses, give rise to larger rock readjustments than expected, supplementary support can then be installed without delay. At Portage Mountain relaxation gauges and drill hole probing proved invaluable for this purpose.

CONCLUSION

Some insight has been given into the importance of geology in the design and construction of underground civil engineering projects. Geological investigations are conducted in the initial phase of any project but due to economic factors the investigations are rarely complete at this stage. The importance of recording and interpreting geological information at all phases during construction is emphasized.

In the design of underground opeings, the science of rock mechanics is a significant complement to geological studies as it provides quantitative data with regard to the effect of forces on rocks. This information reduces the number of calculated risks that must be taken and allows certain techniques such as rock bolting to be better understood.

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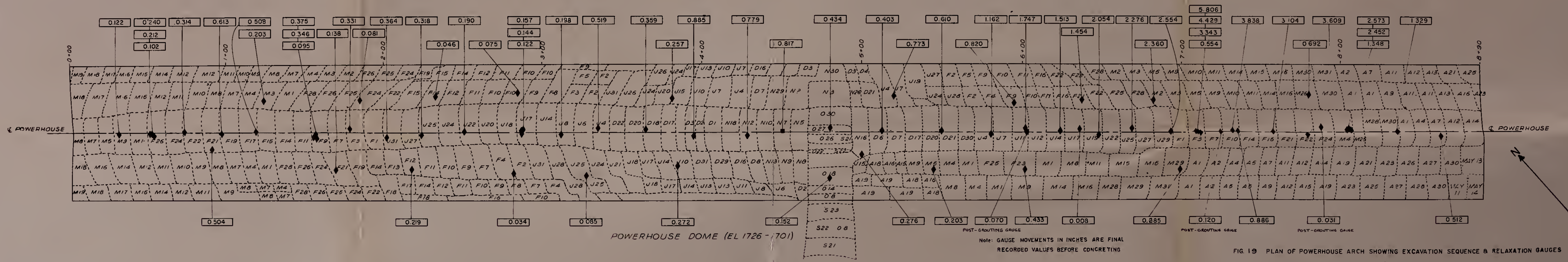
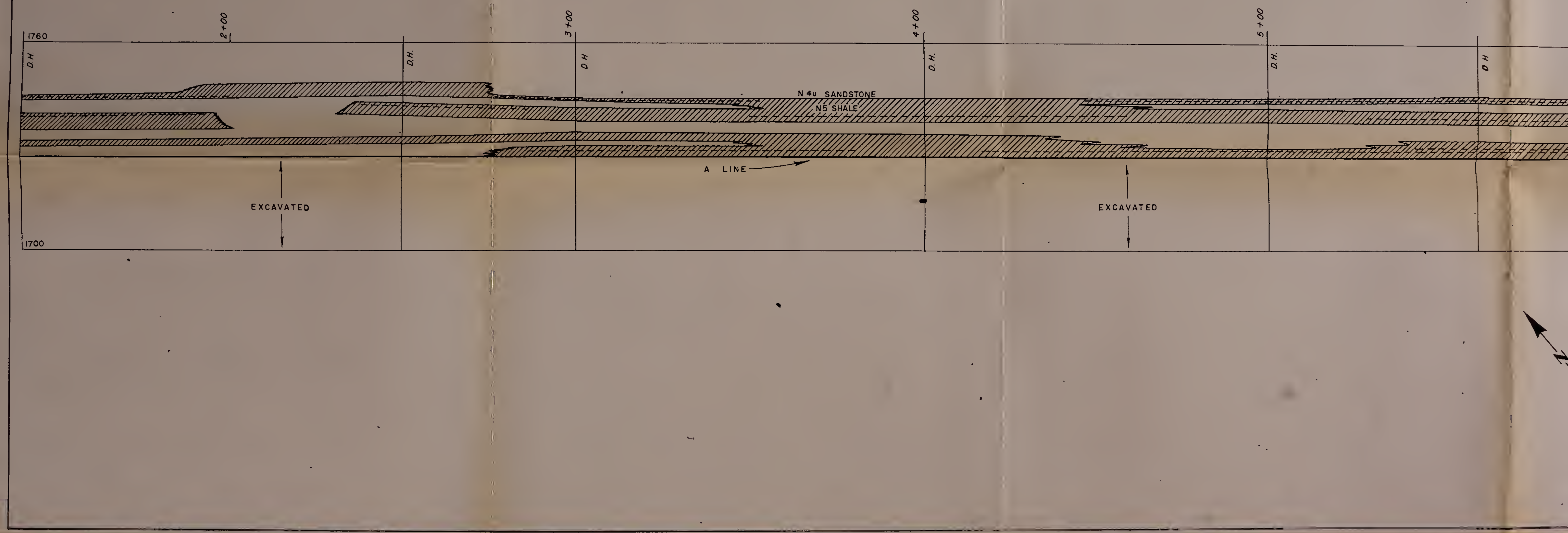
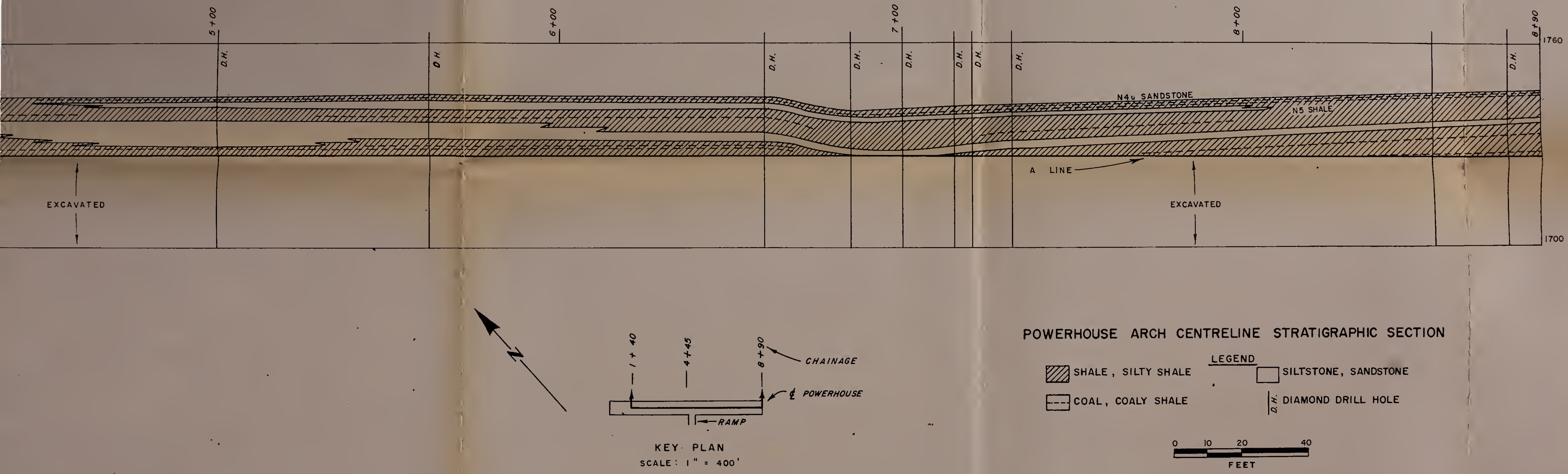


FIG. 19 PLAN OF POWERHOUSE ARCH SHOWING EXCAVATION SEQUENCE & RELAXATION GAUGES

Scale 1" = 40 ft

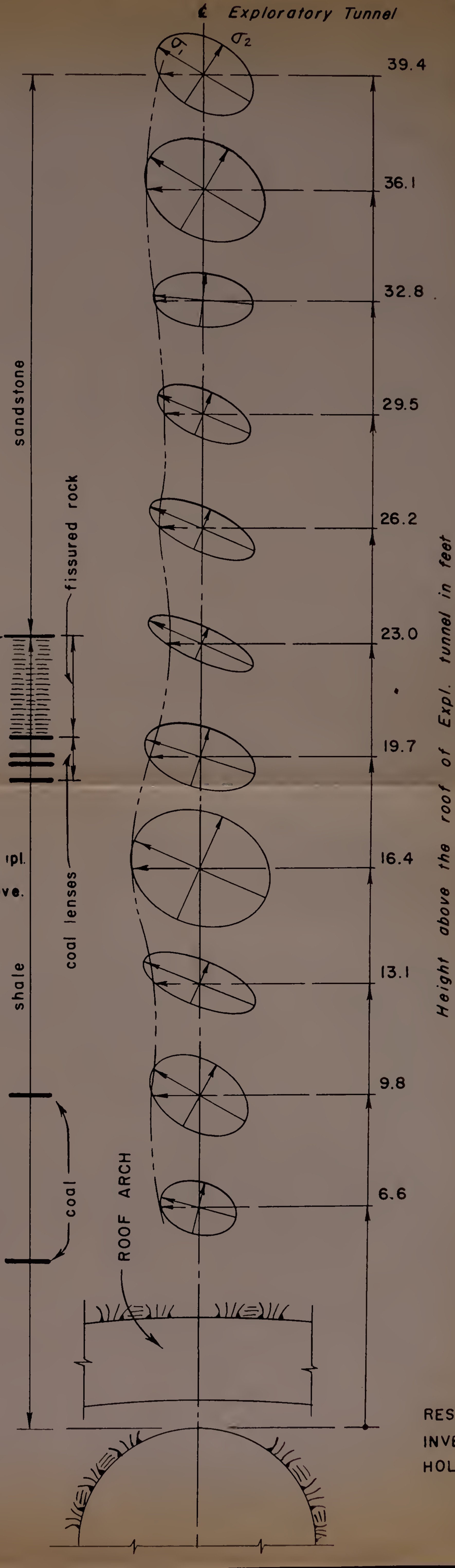


1760
1700
2+00
3+00
4+00
5+00
D.H.
D.H.
D.H.
D.H.
D.H.
D H
N 4u SANDSTONE
N5 SHALE
A LINE
EXCAVATED
EXCAVATED



POWERHOUSE ARCH CENTRELINE STRATIGRAPHIC SECTION

FIG. 30



NOTE:

Stress Ellipses are measurements in horizontal pl. and are viewed from above.

Stress scale: 1" = 3500 psi

Drill Hole Scale: 1" = 3 ft.

RESULT of HAST INVESTIGATIONS HOLE M. P. 4

FIG. 10